
HE WHO SEEKETH KNOWLEDGE FINDETH LIFE
FRANK·A·RANDALL


NIL EXTRA NUMERUM

AMERICAN CONCRETE INSTITUTE

PROCEEDINGS

OF THE

TWENTIETH ANNUAL CONVENTION

Held at Chicago, Ill.

February 25, 26, 27 and 28, 1924.

VOLUME XX

PUBLISHED BY THE INSTITUTE

1807 EAST GRAND BOULEVARD, DETROIT, MICH.

1924

CONTENTS.

AMERICAN CONCRETE INSTITUTE.

OFFICERS.

President:

A. E. LINDAU *(Term Expires 1925 Convention).*

Vice-Presidents:

E. D. BOYER. *(Term Expires 1925 Convention)*

M. M. UPSON. *(Term Expires 1926 Convention).*

Secretary and Treasurer:

HARVEY WHIPPLE, 1807 E. Grand Boulevard, Detroit, Mich.

Directors:

First District.
C. E. NICHOLS *(1926).*

Second District.
N. M. LONEY *(1926).*

Third District.
S. C. HOLLISTER *(1926).*

Fourth District.
J. J. EARLEY *(1925).*

Fifth District.
D. A. ABRAMS *(1925).*

Sixth District.
ARTHUR BENT *(1926).*

Past Presidents:

RICHARD L. HUMPHREY.
LEONARD C. WASON.
WILLIAM K. HATT.
HENRY C. TURNER.
WILLIAM P. ANDERSON

TECHNICAL COMMITTEES

(As of Feb. 25, 1924.)

G-2.—Advisory.

S. C. Hollister, *Chairman.*
Harvey Whipple, *Secretary.*
A. B. Cohen, E. J. Moore, R. F. Havlik, C. E. Lindsley, N. M. Loney.

G-3.—Form of Standards.

W. S. Thomson, *Chairman.*
R. W. Boyd, *Secretary.*
Frank C. Wight.

G-4.—Nomenclature.

W. A. Slater, *Chairman.*
F. A. Hitchcock, *Secretary.*
A. B. McDaniel.

G-5.—Wason Medal.

Frank C. Wight, *Chairman.*
J. C. Pearson, *Secretary.*
A. R. Lord, Cloyd Chapman, W. A. Slater, C. E. Nichols.

J-1.—Joint Committee on Concrete and Reinforced-Concrete—Institute Representation.

S. C. Hollister, *Chairman.*
A. B. McMillan, R. W. Lesley, H. D. Loring, E. J. Moore.

J-2.—Institute Representation on Joint Culvert Pipe Committee.

B. S. Pease, *Chairman.*
A. B. Cohen.

E-1.—Reinforced-Concrete Building Design and Specifications.

A. R. Lord, *Chairman.*
J. A. Ferguson, *Secretary.*
A. E. Lindau, George E. Horr, Manton E. Hibbs, A. W. Stephens, F. R. McMillan, H. N. Howe, H. P. Mueller, W. A. Slater, R. B. Young, H. M. Westergaard, H. F. Gonnerman, Carl Preis, F. E. Richart, S. C. Hollister, H. J. Brunnier, R. L. Bertin, H. D. Loring, D. A. Abrams, F. C. Wight.

E-4.—Fire Resistance of Concrete.

N. D. Mitchell, *Chairman.*
J. R. Dwyer, *Secretary.*
Leslie H. Allen, M. J. O'Brien, Leonard C. Wason.

E-5.—AGGREGATES.

H. S. MATTIMORE, *Chairman.*

R. W. CRUM, *Secretary.*

F. R. MCMILLAN, R. L. BERTIN, J. G. AHLERS, N. M. LONEY, DUFF A. ABRAMS, C. S. WICKER, H. J. LOVE, L. R. FERGUSON, H. E. BREED, H. F. CLEMMER.

E-6.—DESTRUCTIVE AGENTS AND PROTECTIVE TREATMENTS.

M. M. UPSON, *Chairman.*

P. J. FREEMAN, *Secretary.*

J. A. FERGUSON, G. E. WARREN, E. H. BARLOW.

E-7.—WATERPROOFING.

S. C. HOLLISTER, *Chairman.*

E. H. BERGER, *Secretary.*

MAXIMILIAN TOCH, DUFF A. ABRAMS, J. C. PEARSON, HERMAN FOUGNER, W. A. FRERET, S. R. CHURCH, M. C. WITHEY, ERNEST ASHTON, G. L. LUCAS, RAYMOND H. KRECKER, C. N. FORREST, A. W. STEPHENS, A. E. HORN, A. B. COHEN, J. H. LIBBERTON, JOSEPH ROSE.

E-8.—EXPANSION JOINTS IN CONCRETE CONSTRUCTION.

LANGDON PEARSE, *Chairman,*

R. R. LEFFLER, *Secretary.*

HENRY F. BRYANT, FREDERICK FAY, JOHN LEONARD VOGEL, E. A. RENWICK, FRANK E. RICHART, A. J. REYNVAAN, STANTON WALKER.

C-1.—CONTRACTORS' PLANT.

J. G. AHLERS, *Chairman.*

W. F. LOCKHARDT, *Secretary.*

J. W. TOWNSEND, W. F. GILCREAST, F. I. GINSBERG, LOUIS H. USILTON, D. L. KNEEDLER, R. E. BROOKS.

C-2.—CONCRETE FLOOR FINISH.

N. M. LONEY, *Chairman.*

A. C. IRWIN, *Secretary.*

J. E. FREEMAN, W. F. KEARNS, CHARLES R. MILLER, T. A. PEARSON, W. M. RYNERSON, A. C. TOZZER, P. A. WELLS, H. R. WOOD.

C-3.—TREATMENT OF CONCRETE SURFACES.

J. C. PEARSON, *Chairman.*

F. A. HITCHCOCK, *Secretary.*

J. G. AHLERS, W. E. BALLINGER, D. K. BOYD, E. D. BOYER, CLOYD M. CHAPMAN, WHARTON CLAY, W. H. COOKMAN, J. J. EARLEY, W. E. EMLEY, J. E. FREEMAN, A. J. GARROD, H. A. GARDNER, C. N. GODFREY, G. E. HORR, M. A. ILLING, J. B. ORR, H. P. WARNER, LINN WHITE, H. WITTEMORE BROWN, W. E. HART, D. F. HOLTMAN.

C-5.—MEASUREMENT OF AND ESTIMATING CONCRETE.

F. R. WALKER, *Chairman.*

WILLIAM F. JENRICK, *Secretary.*

OTTO WOLPERT, C. M. CHUCKROW, W. F. TUBESING, H. W. HANLEY, J. W. GINDER, H. J. GOULD, A. E. HORST, WM. GRAVES SMITH, F. H. HUNTER, A. W. STEPHENS, J. G. AHLERS, CHARLES F. DINGMAN.

C-6.—FIELD METHODS.

R. L. BERTIN, *Chairman.*

R. B. YOUNG, *Secretary.*

DUFF A. ABRAMS, GEORGE E. HORR, F. R. MCMILLAN, H. S. MATTIMORE, P. A. WELLS, N. M. LONEY, W. A. SLATER.

S-1.—REINFORCED-CONCRETE CHIMNEYS.

ERIC PLAGWIT, *Chairman.*

A. C. IRWIN, *Secretary.*

EDWARD GODFREY, JOHN LOWELL, I. F. STERN, E. A. DOCKSTADER, H. C. BLAKE.

S-2.—REINFORCED-CONCRETE HIGHWAY BRIDGES AND CULVERTS.

A. B. COHEN, *Chairman.*

A. C. IRWIN, *Secretary.*

J. H. AMES, G. F. BURCH, R. L. COCHRAN, A. B. FLETCHER, C. B. MCCULLOUGH, C. T. MORRIS, J. W. MUELLER, W. J. TITUS, C. M. UPHAM, G. G. WICKLINE, G. L. GROVER, E. F. KELLEY.

S-3.—REINFORCED AND PLAIN CONCRETE SEWERS AND CONDUITS.

LANGDON PEARSE, *Chairman.*

M. W. LOVING, *Secretary.*

W. L. LEA, F. A. MARSTON, C. J. REQUARDT, C. E. DE LEUW, ASA E. PHILLIPS, E. S. RANKIN.

S-4.—CONCRETE STORAGE TANKS.

G. A. SMITH, *Chairman.*

W. E. HART, *Secretary.*

S. C. HOLLISTER, R. E. WILSON, G. W. VANDYKE, R. P. DURHAM, J. V. SCHAEFER, W. S. HEWETT.

S-5.—CONCRETE HOUSES.

JOHN A. FERGUSON, *Chairman.*

A. C. IRWIN, *Secretary.*

LESLIE H. ALLEN, KATE GLEASON, PAUL HUEBER, J. C. PEARSON, E. G. PERROT, R. F. HAVLIK, RUSSEL BARR WILLIAMSON, J. W. LOWELL, J. E. LAMBIE, FRED W. LUMIS, JOHN H. MCCLATCHY, C. G. DUNNELLS, WILLIAM GOLDIE, JR., F. M. MCCULLOUGH, R. K. MCGEE.

S-6.—CONCRETE ROADS AND PAVEMENTS.

W. M. ACHESON, *Chairman.*

C. R. EGE, *Secretary.*

H. E. BREED, B. S. PEASE, W. A. MCINTYRE, H. E. HILTS, C. V. R. FULLENWIDER.

P-1.—STANDARD BUILDING UNITS.

W. R. HARRIS, *Chairman.*

E. W. DIENHART, *Secretary.*

NEWTON BENSON, W. H. CAREY, AUSTIN CRABBS, R. F. HAVLIK, H. K. HOLSMAN, F. J. KINSINGER, C. E. LINDSLEY, C. A. STEWARD, STANTON WALKER, H. G. DAVIS, J. W. LOWELL, EDGAR BARNETT, J. A. FERGUSON, M. W. PLUMB, WM. GOLDIE, JR., J. J. BLISS, J. H. MCCLATCHY, HENRY BOETTCHER, J. W. OEHMANN, CLOYD M. CHAPMAN.

P-4.—CONCRETE STAVES.

W. O. BRASSERT, *Chairman.*

W. G. KAISER, *Secretary.*

C. C. WOODY, KENT MERRY, L. H. DEMUTH.

P-5.—FIRE RESISTANCE OF CONCRETE BUILDING UNITS.

LESLIE H. ALLEN, *Chairman.*

HARVEY WHIPPLE, *Secretary.*

W. R. HARRIS, S. H. INGBERG, H. J. LOVE, F. W. LUMIS, A. R. SMALL, G. M. FRIEL, STANTON WALKER, JOHN H. MCCLATCHY, AUSTIN CRABBS, J. J. BLISS.

P-6.—CONCRETE PRODUCTS PLANT OPERATION.

J. W. LOWELL, *Chairman.*

E. W. DIENHART, *Secretary.*

W. D. M. ALLAN, A. G. SWANSON, JACOB BOSCH, G. J. LENGST, J. J. BLISS, GEORGE SAFFERT, DAVIS EWING, W. H. CAREY, AUSTIN CRABBS, H. F. GONNERMAN, C. E. LINDSLEY.

P-7.—PIPE, DRAIN TILE AND CONDUIT.

C. F. BUENTE, *Chairman.*

M. W. LOVING, *Secretary.*

ERNEST F. BENT, M. L. ENGER, J. E. LONGLEY, R. S. LANDER, J. A. DUNN, C. H. BULLEN, SAMUEL FORTIER, PAUL KIRCHER, B. S. PEASE, WILLIS WHITED, J. S. LAMBIE.

P-8.—CONCRETE ROOF TILE.

P. J. FREEMAN, *Chairman.*

F. W. IGGBERG, *Secretary.*

M. G. WATKINS, P. J. MARTIN, W. ROOPE, J. R. DWYER, A. E. WITHEROW, RAYMOND WEEKS, G. R. MILLS, OTTO WALTER, R. L. MORRISON, J. C. WITT, L. H. ALLEN, H. A. BARNESKAN, THOMAS G. MOORE, H. F. GONNERMAN.

BY-LAWS.

AMERICAN CONCRETE INSTITUTE.

Article I.

MEMBERS.

Section 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of the Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

Sec. 2. A firm or company shall be treated as a single member.

Sec. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

Sec. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener, if necessary, to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at the time with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

Sec. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

Article II.

OFFICERS.

Section 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

Sec. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

Sec. 3. There shall be a Committee of five members on Nomination of Officers, elected by letter ballot of the members of the Institute, which is to be canvassed by the Board of Direction on or before September 1 of each year.

The Committee on Nomination of Officers shall elect by letter ballot of its members, candidates for the various offices to become vacant at the next Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least 60 days prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within 20 days thereafter. The consent of all candidates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted 30 days before the Annual Convention to the members of the Institute for letter ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

Sec. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-President and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election a President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty of any officer of the Institute, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

Sec. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election, appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

SEC. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

SEC. 7. The Board of Direction shall appoint a Committee on Resolutions, to be announced by the President on the first regular session of the annual convention.

SEC. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

SEC. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

SEC. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

SEC. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and furnish such bond as may be determined by the Board of Direction.

SEC. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

SEC. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

ARTICLE III.

MEETINGS.

SECTION 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of Convention.

SEC. 2. The Board of Direction shall meet during the Convention at which it is elected, effect organization and transact such business as may be necessary.

SEC. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

SEC. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

ARTICLE IV.

DUES.

SECTION 1. The fiscal year shall commence July 1st.

SEC. 2. The annual dues shall be ten dollars ($10.00) payable annually in advance from first of the month following notification of the applicant of his election by the Board of Direction.

SEC. 3. Each member shall be entitled to receive one copy of one volume of the Proceedings for each membership year and additional volumes at a price fixed by the Board of Direction.

SEC. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon payment of all indebtedness against them upon the books of the Institute.

ARTICLE V.

STANDARDS.

SECTION 1. Proposed new or revised Standard Specifications, Standard Practice and Standard Definitions, when approved by a majority voting in the committee in which they originate, shall be submitted in the form adopted in the Standard Form of Standards to the secretary of the Institute 60 days prior to the opening of the annual convention at which they are to be presented. The secretary of the Institute shall cause these proposed new standards or revised standards to be printed as Proposed Tentative Standards and mailed to the full membership of the Institute thirty days prior to the opening of the convention. As there amended and approved, they shall be published in the Annual Proceedings, next issued as Tentative Standards. At a subsequent annual convention they may again be offered unamended, by their originating committees as proposed standards, and as there approved by a majority of those voting, they shall be submitted to letter ballot of the Institute membership, to be canvassed within ninety days thereafter. Such proposed standards shall be considered adopted unless at least 10 per cent of the total membership shall vote in the negative.

ARTICLE VI.

AMENDMENT.

SECTION 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF PROCEEDINGS OF THE TWENTIETH ANNUAL CONVENTION.

Drake Hotel, Chicago, Illinois.

FIRST SESSION, MONDAY, FEBRUARY 25, 1924, 2 P. M.

The convention was called to order by William P. Anderson, president of the American Concrete Institute.

The following papers were read and discussed:

"How Structures Withstood the Japanese Earthquake and Fire," by H. M. Hadley.

"Effect of Japanese Earthquake and Fire on Structures," by Joseph S. Ruble.

"Old and New Methods of Constructing Concrete Bridges," by J. B. Hunley.

JOINT SECOND SESSIONS, MONDAY, FEBRUARY 25, 1924, 8 P. M.

Two co-ordinate sessions were held on Monday evening (a) for construction superintendents, with Vice-President M. M. Upson in the chair, and (b) for concrete products manufacturers, with A. J. R. Curtis in the chair. The meeting consisted of the discussion of certain questions previously printed.

THIRD SESSION, TUESDAY, FEBRUARY 26, 1924, 9.30 A. M.

Past-President Richard L. Humphrey in the chair.

The report of Committee S-6, Concrete Roads and Pavements, was presented by its chairman, W. M. Acheson.

The report of the committee consisted in the presentation of

(1) Proposed Standard Specifications for Portland Cement Concrete Pavements. (One-Course Portland Cement Concrete Pavement for Highways).

(2) Proposed Tentative Specifications for Portland Cement Concrete Pavements.

- (a) Two-Course Portland Cement Concrete Pavement for Highways.
- (b) One-Course Portland Cement Concrete Street Pavement.
- (c) Two-Course Portland Cement Concrete Street Pavement.

With certain amendments noted in the reprint of the specifications in this volume, specification (1) was approved and sent to letter ballot for adoption as standard specification and specifications (2) were accepted as tentative.

The following paper was read and discussed:

> "New Practice of Iowa State Highway Department for Control of Quality of Concrete," by R. W. Crum.

The report of the Joint Committee on Culvert Pipe, Committee J-2, was presented by its chairman, B. S. Pease. This was merely a progress report. The report was received and the recommendation passed that the Institute continue to be represented on the Joint Committee with the approval of the Board of Direction.

The report of Committee P-7, Concrete Pipe, Drain Tile and Conduits, was presented by its chairman, C. F. Buente. The report consisted in the presentation of the proposed tentative specifications:

(1) Concrete Drain Tile

(2) Plain Concrete Sewer Pipe

(3) Reinforced-Concrete Sewer Pipe

With the amendments noted in the full statement of the report in this volume, the specifications were accepted as tentative.

The report of Committee C-1, Contractors Plant, was presented by its chairman, J. G. Ahlers. The report consisted in the presentation of a report entitled "The Selection and Design of Contractors Plant."

The report of Committee C-5, Estimates and Estimating, was presented by its chairman, Frank R. Walker. It was a progress report and was accepted.

The following paper was read and discussed:

> "The Economic Value of Admixtures," by J. C. Pearson and Frank A. Hitchcock.

The report of Committee C-2, Concrete Floor Finish, was presented by its chairman, N. M. Loney. The report consisted in the submission of Proposed Standard Specifications for Concrete Floors. This was adopted and sent to letter ballot for adoption as a standard of the Institute.

The report of Committee S-3, on Concrete Sewers, was presented by its chairman, Secretary M. W. Loving. This report consisted in the submission of Proposed Standard Specifications for Concrete Sewers. This was adopted and sent to letter ballot for adoption as a standard of the Institute.

Committee C-2, through its chairman, N. M. Loney, submitted a revision of the existing standard for Portland Cement Concrete Sidewalks. With the amendments noted in this volume, the specification was adopted as a tentative standard.

On February 26, 1924, at 2 P. M., there was a demonstration of the operation and manufacture of trim stone and ornamental concrete conducted by R. F. Havlik and others.

FOURTH SESSION, TUESDAY, FEBRUARY 26, 1924, 8 P. M.

Past-president Richard L. Humphrey in the chair.

The report of Committee P-1, Standard Building Units, was presented by its chairman, Wallace R. Harris. The report consisted in the presentation of revised Tentative Standard Specifications for Concrete Building Block and Concrete Building Tile and Proposed Tentative Standard Specifications for Concrete Brick. All were adopted by the meeting.

The report of Committee P-4, Concrete Staves, was presented by its secretary, W. G. Kaiser. The proposed Tentative Standard Specifications on Concrete Staves was adopted as a tentative standard.

The following papers were read and discussed:

"Building Codes and Their Relation to the Concrete Products Industry," by William F. Lockhardt.

"Building Fireproof Homes," by Paul Hueber.

The report of Committee S-5 on Concrete Houses was presented by its chairman, J. A. Ferguson. It was presented as a progress report and with discussion was accepted.

The report of Committee P-5, Fire Resistance of Concrete Building Units, was presented by its chairman, Leslie H. Allen. The report was a continuation of the report submitted last year on fire tests of blocks. The report was accepted.

The report of Committee G-4 on Nomenclature was presented by W. A. Slater, chairman. This was a progress report and was accepted.

TWENTIETH ANNIVERSARY DAY

FIFTH SESSION, WEDNESDAY, FEBRUARY 27, 1924, 10 A. M.

This day was set aside to celebrate the growth of the Institute and the tremendous strides the concrete industry has made in the twenty-year period. The program consisted in the presentation of papers as follows:

(1) Opening address by Richard L. Humphrey.

(2) "Cement is the Magic of Concrete," by F. W. Kelley.

(3) "Concrete Roads," by A. N. Johnson.

(4) "Mass Concrete," by Arthur P. Davis (paper read by A. M. Stephens).

(5) "Mechanical Equipment for Handling Concrete," by A. W. Ransome.

SIXTH SESSION, WEDNESDAY, FEBRUARY 27, 1924, 2.30 P. M.

Past-president Richard L. Humphrey in the chair.

(6) "Reinforced-Concrete Buildings," by Albert Kahn.

(7) "Concrete Bridges," by A. E. Lindau.

(8) "Concrete for Foundations and Water Front Work," by M. M. Upson.

(9) "Concrete Products," by R. F. Havlik.

(10) "Architectural Concrete," John J. Earley.

(11) "Engineering Research," by A. N. Talbot.

(12) "Making Good Concrete," by Duff A. Abrams.

(13) "The Promise of Future Development," by Richard L. Humphrey.

On the evening of Wednesday, February 27, a banquet was held.

SEVENTH SESSION, THURSDAY, FEBRUARY 28, 1924, 9.30 A. M.

Vice-President A. E. Lindau in the chair.

The following papers were read and discussed:

"What Engineers and Contractors May Expect to Accomplish with High Alumina Cement," by Henry S. Spackman.

"Field Tests of Concrete," by J. G. Ahlers and Stanton Walker.

"Methods Used and Results Obtained in Control of Concrete in Building Construction," by W. A. Slater.

The report of Committee C-6, Field Methods, was presented by R. L. Bertin, chairman. It consisted in a paper entitled "Suggestions for the Production of Better Concrete."

BUSINESS SESSION

The report of the Board of Direction was read by the secretary.

The following by-laws were accepted and passed to letter ballot of the membership:

ARTICLE V (*as it stood*)

RECOMMENDED PRACTICE AND SPECIFICATIONS

Section 1. Proposed Recommended Practice and Specifications to be submitted to the Institute must be mailed to the members at least thirty days prior to the Annual Convention, and as there amended and approved, passed to letter ballot, which shall be canvassed within sixty days thereafter, such Recommended Practice and Specifications shall be considered adopted unless at least 10 per cent of the total membership shall vote in the negative.

ARTICLE V (*as adopted*)

Section 1. Proposed new or revised Standard Specifications, Standard Practice and Standard Definitions, when approved by a majority voting in the committee in which they originate, shall be submitted in the form adopted in the Standard Form of Standards to the secretary of the Institute 60 days prior to the opening of the annual convention at which they are to be presented. The secretary of the Institute shall cause these proposed new standards of revised standards to be printed as Proposed Tentative Standards and mailed to the full membership of the Institute thirty days prior to the opening of the convention. As there amended and approved, they shall be published in the Annual Proceedings, next issued as Tentative Standards. At a subsequent annual convention they may again be offered unamended, by their originating committees as proposed standards, and as there approved by a majority of those voting, they shall be submitted to letter ballot of the Institute membership, to be canvassed within ninety days thereafter. Such proposed standards shall be considered adopted unless at least 10 per cent of the total membership shall vote in the negative.

President W. P. Anderson in the chair.

The report of the Board of Tellers announced the election of the following officers:

President, A. E. Lindau, of Chicago.

Vice-President (2-year term), M. M. Upson, New York.

Treasurer, Harvey Whipple, Detroit.

Directors, 1st District, C. E. Nichols, Boston.
2d District, E. D. Boyer, New York.*
6th District, Arthur Bent, Los Angeles.

---

*Because of the elevation of A. E. Lindau from the office of vice-president to that of president, the Board of Direction, in accordance with the By-Laws, filled the position of vice-president left vacant. E. D. Boyer was made vice-president, and N. M. Loney, N. Y. director to fill the place left vacant by Mr. Boyer in the 2d district.

EIGHTH SESSION, THURSDAY, FEBRUARY 28, 1924, 2 P. M.

W. P. Anderson in the chair.

The report of Committee P-8, Expansion Joints, was presented by its chairman, Langdon Pearse. The paper consisted in a report on expansion joints in concrete construction. It was accepted.

The report of Committee E-5, Aggregates, was presented by its secretary, R. B. Crum. It was a progress report and was accepted.

The report of Committee E-6, Concrete Failures, was presented by its chairman, M. M. Upson. This consisted in the presentation of studies of two concrete failures, one by P. J. Freeman and the other by John A. Ferguson.

The report of the Committee E-7, Waterproofing, was presented by its chairman, S. C. Hollister. This was a progress report and was accepted.

The following paper was read and discussed:

"Bending Moments in Columns," by F. E. Richart.

The report of Committee E-1, Reinforced-Concrete Building Design and Specifications, was presented by its chairman, A. R. Lord. It consisted of a progress report, which was accepted.

THE WASON MEDAL.

AWARDED EACH YEAR TO THE AUTHOR OF THE MOST MERITORIOUS PAPER PRESENTED TO THE PREVIOUS ANNUAL CONVENTION.

Awarded 1924 to

J. J. EARLEY, for his paper, "Building the Fountain of Time," Presented to the 1923 Convention.

PREVIOUS AWARDS.

1916 Convention Paper—A. B. McDANIEL, "Influence of Temperature on the Strength of Concrete."

1917 Convention Paper—CHARLES R. GOW, "History and Present Status of the Concrete Pile Industry."

1918 Convention Paper—DUFF A. ABRAMS, "Effect of Time of Mixing on the Strength and Wear of Concrete."

1919 Convention Paper—W. A. SLATER, "Structural Laboratory Investigations in Reinforced Concrete Made by Concrete Ship Section, Emergency Fleet Corporation."

1920 Convention Paper—W. A. HULL, "Fire Tests of Concrete Columns."

1921 Convention Paper—H. M. WESTERGAARD, "Moments and Stresses in Slabs."

1922 Convention Paper—GEORGE E. BEGGS, "An Accurate Mechanical Solution of Statically Indeterminate Structures by Use of Paper Models and Special Gauges."

Papers Delivered at the Twentieth Anniversary Session

of the

American Concrete Institute

February 27, 1924.

The Progress of Two Decades.

By Richard L. Humphrey

Consulting Engineer, Philadelphia, Pa., Past-President, American Concrete Institute.

WHILE this is its Twentieth Annual Convention, the American Concrete Institute is but a little over nineteen years old.

Twenty years is an almost negligible portion of time in the progress of civilization, although it is about a third of the life of the average man.

The interval that has elapsed since the formation of this Institute seems but as yesterday and yet during that time, brief though it is, a most tremendous development of the cement industry has taken place.

On an occasion of this kind it is fitting in the opening address to briefly review the progress of the art before taking up the particular developments in the industry during the life of the Institute.

The use of calcareous materials as a binder in masonry probably began more than five centuries before the Christian era.

The ruins of massive structures built by the Ancient Egyptians reveal the use of mortar in the joints of these edifices.

Pliny and Vitruvius have recorded the various methods of application used by the ancient Romans. The former ascribed the good condition of the mortar to the ancient laws which prohibited contractors from using lime until it had slaked for three years.

It was the Roman practice to use fine particles of burnt earth or volcanic cinders coming from Pozzuoli at the foot of Vesuvius and they appeared to have discovered the fact that this material rendered mortar capable of hardening under water.

Pliny further tells us that the Roman mortars were very bad and that;

> "The cause which makes so many houses fall in Rome resides in the bad quality of the cement."

Literature seems to indicate that following the downfall of the Roman Empire, Roman cement fell into disuse.

Roman mortars and concretes have survived the ravages of time by reason of the favorable climatic conditions to which they were exposed. A great deal of the strength of these ancient mortars is attributable to their gradual induration with the ages. The Cleopatra obelisk in Central Park, New York, that had stood for centuries in Egypt, began to disintegrate under the action of freezing and thawing, and had it not been protected from such action would not long have survived the rigorous climate of the new world.

The survival of Roman mortars and concretes for several thousand years gave them a reputation for durability which led early experimenters of the eighteenth century to seek to recover the lost Roman Art.

In 1780 Dr. Bryan Higgins presented his "Experiments and Observations, made with the View of Improving the Art of Composing and Applying Calcareous Cements and of Preparing Quick-Lime: Theory of these Arts; and Specifications of the Authors cheap and durable Cement, for Building, Incrustation or Stuccoing, and Artificial Stone."

In this book he called attention to the fact that,—

> "Where the weather is so variable and trying, and the mortar commonly used is so bad, that the timbers of houses last longer than the walls, unless the mouldering cement be frequently replaced by pointing,"

and states that,—

> "I resolved in the beginning of the year 1775 to investigate more closely than I had hitherto done, the principles on which the induration and strength of calcareous cements depend; not doubting that this would lead me by an untried path to recover or to excel the Roman cement, which in aqueducts and the most exposed structures has withstood every trial of fifteen hundred or two thousand years."

Dr. Higgins secured an exclusive right for the manufacture of his cement through letters patent dated January 8, 1779.

The successful efforts of John Smeaton in 1756 to secure a suitable material that would harden under water, gave fresh impulse to the development of the art. In his "Narrative of

the Building etc., of the Eddystone Lighthouse," published in 1791 he records his important experiments on hydraulic mortars. He found that some limes were better than others and states that he,—

> "was very desirous to get some light into some of the sensible qualities that might properly occasion the difference, or at least be a mark of distinction."

he further states that,—

> "the fitness of lime for water building seems neither to depend upon the hardness of the stone, the thickness of the stratum, nor the bed or matrix in which it is found, nor merely on the quality of clay it contains, but in burning and falling down into a powder of a buff colored tinge, and in containing a considerable quantity of clay, I have found all the water limes to agree."

The patent (the first for the manufacture of Cement) granted to James Parker in 1796 was for an invention consisting of reducing to powder natural nodules of agrillaceous and calcareous composition and using with water to form a mortar or cement stronger and harder than any mortar or cement then prepared by artificial means. Because of the reputation of ancient mortars this material was known eventually as Roman Cement.

Collet-Descotils, a professor in the Ecole des Mines called attention in 1813 to the fact that the silica found in limestone which is insoluble in acids became soluble when the limestone was calcined at a red heat and that this was the reason for the hydraulic properties of cement.

The James Frost patent of 1822 was for the manufacture of a new cement or artificial stone prepared by calcining in a kiln a mixture of limestone or marls, free from alumina or argillaceous earth, but containing from 9 to 40 per cent of silicious earth or silica and reducing the resulting mass to a powder.

The celebrated researches of L. J. Vicat, a French engineer. on building lime, concrete and ordinary mortars, published in 1818, had a widespread influence; and in that year St. Ledger was granted a French patent for making hydraulic lime by calcining a mixture of chalk and clay. All these investigations were leading to the development of a suitable hydraulic building material.

It finally remained for John Aspdin, a bricklayer of Leeds, England, to give the material a definite status and a name (in his patent it was called "Portland Cement" from its resemblance to the building stone from the Isle of Portland) through his patent which he designated as

> "An Improvement in Modes of Producing an Artificial Stone"

and wherein he described a method of making a cement or artificial stone for stuccoing buildings, waterworks, cisterns, or any other purpose to which it may be applicable; he made a specific mixture of limestone and clay which he calcined in a kiln until the carbonic acid was entirely expelled; it was then ground to a powder and the powder was mixed with water to the consistency of mortar and applied to the purposes required.

In 1825 Aspdin established a factory at Wakefield, England. 1828 Aspdin cement was used in the Thames Tunnel.

The manufacture of Portland Cement was well established in England by 1850 and was well under way in Continental Europe in 1855.

Natural cement was first made in this country in 1818, but it was not until 1872 that Portland Cement was produced at Coplay, Pa., by David O. Saylor.

Natural cement reached its maximum production in 1899, after which Portland Cement, because of its superior hydraulic qualities and strength and the economy resulting from its use, began to supplant it as a building material. The use of Portland Cement increased, until in 1923 the consumption in this country reached the undreamed total of more than one hundred and thirty million barrels.

The most important factor in the development of the Portland Cement industry was the introduction of the rotary kiln; although patented by Frederick Ransome in England in 1885 it was developed and perfected in this country.

Until the advent of this kiln cement was manufactured in vertical or shaft kilns. These were economical as regards fuel but required a great deal of labor in handling the clinker. Since fuel was relatively cheap and labor expensive the rotary kiln was found to be the more economical in this country and it was also found

that the resulting product was more uniform in quality. With increasing labor costs in Europe, the home of the vertical kiln, the use of the rotary kiln gradually increased, and now its use is good practice all over the world.

The element of suspicion as to quality found expression at the meetings of the Engineers' Club in Philadelphia in the early nineties, where it was predicted that concrete made with rotary kiln cement would disintegrate in five years.

Skepticism has accompanied all the various uses and applications of cement and concrete and is no new thing in this industry. But in spite of skepticism it gradually developed, and grew into a great industry.

The history of the development of the Art of making cement has been a most interesting one, especially as regards its application in building; the embedment of metal to greatly increase its resistance of tensile stresses, gave to the material an unbelievable applicability.

In 1850 Joseph Gibbs obtained a British patent for casting solid walls of concrete in wooden forms.

Although Joseph Monier, a French gardener, used a reinforcing frame work or skeleton of metal in the construction of concrete garden pots, tubs and tanks, his experiments were antedated by Lambot, who constructed a small reinforced-concrete boat in 1850 which he exhibited at the Paris exposition in 1855; in the same year Coignet announced his principles for reinforcing and proposed the construction of beams, arches, pipes, etc.

Both Coignet and Monier had exhibits at the Paris Exposition in 1867. In that year Monier took out his first patent on reinforcement, principally two-way reinforcement of slabs, but his system was not put into general use until after 1880, although he exhibited it at the Antwerp Exposition in 1879.

In 1884-5 German-American rights were acquired by German engineers; one of these, G. A. Wayss, and John Bauschinger published in 1887 the results of their experiments and investigations of the Monier system.

The work of Thaddeus Hyatt, an American who made beam tests in Kirkaldy's laboratory in England in 1876 and 1877, had a pioneering influence on the development of the art. Hennebique built reinforced-concrete floors as early as 1879. It was not, how-

ever, until 1892 that he obtained a patent for his system of reinforced-concrete construction.

Melan originated his system in the 90's.

Between 1891 and 1894 Moeller in Germany, Meunsch in Hungary and Melan in Austria were pioneers in the development of the art of reinforced-concrete construction in Europe.

In 1866 C. S. Hutchinson obtained a United States patent for hollow block and brick walls which formed continuous vertical air spaces. In 1874 T. B. Rhodes also obtained a United States patent for shingles and hollow blocks.

Patents were granted in this country to C. W. Stevens for the manufacture of artificial stone by the sand mold process, and to J. C. McClenahan for artificial stone, in 1902.

The first wholly reinforced-concrete structure was the W. E. Ward house in New York erected in 1875.

The first concrete bridge in this country was built in Prospect Park, Brooklyn, N. Y., in 1871. The Erie Railroad is said to have lined its first tunnel with concrete in 1874.

Messrs. H. V. Jackson, G. W. Perry and E. L. Ransome made actual demonstrations of Reinforced-Concrete on the Pacific Coast as early as 1877.

Mr. Ernest L. Ransome took out his first patent in 1882 and constructed his first reinforced-concrete Building, in 1884, in which year he made experiments on reinforced-concrete beams and received a patent on his system of floor construction.

In 1905, when this Institute had its birth Reinforced-Concrete was just beginning to be used to a considerable extent as a structural material for buildings and other structures.

The effort to introduce this material was resisted by those who raised doubts as to the safety of such construction; the building codes were unfavorable and the cost of construction so high as to make it at first unattractive to the investor. Concrete on the other hand was considered very satisfactory for side walks, and massive masonry foundations.

The American Society of Civil Engineers had appointed a Committee on A Uniform System of Tests of Cement which presented a report in 1885 and the methods of tests therein recommended served as a basis for specifications until 1896 when the speaker inspired a series of editorials in the Engineering Record

which called attention to the inadequacy of the 1885 rules and led to the appointment in 1897 by the Society of a new "Committee on the Proper Manipulation of the Tests of Cement."

In 1898 the American Society for Testing Materials appointed a Committee on Standard Specifications for Cement and in 1912 there was created from among the several departments of the Government, the Departmental Conference.

> The recommendations of these Committees differed and "in order to secure uniformity in specifications for Cement" Committees representing the three organizations met in October, 1912, and formed the Joint Conference on Uniform Methods of Tests and Standard Specifications for Cement."

This Conference Committee presented its report to its constituent organizations in June, 1916.

In time the specification resulting from this Conference was adopted with slight modification as the American Standard Specification for Portland Cement.

You are familiar with the years of effort of these Committees and it is not necessary to burden you with their detail.

The desire expressed in the above mentioned effort to secure proper workmanship and materials in building construction, was prevalent also in days of ancient Rome.

Vitruvius writing in the first century after Christ describes in his work on Architecture how to make concrete pavements and floors and walls around wells.

He specified that,—

> "In the first place the purest roughest sand that can be had is to be procured; then material is to be prepared of broken flint, where no single piece is to weigh more than one pound; the lime must be very strong, and in making it into mortar, five parts of sand are to be added to two of lime; the flint work is combined with the mortar, and of it the walls in the excavation are brought up from the bottom and rammed with wooden bars covered with iron."

When this Institute was organized the first Joint Committee on Concrete and Reinforced-Concrete had just been organized

(summer of 1904) and was beginning the very important work of preparing its recommendations for these materials which were to serve as a basis for the specifications that have been prepared by the Second Joint Committee, the Joint Committee on Standard Specifications for Concrete and Reinforced-Concrete.

The cast iron molding machine for forming cement mortar blocks for building purposes was being extensively developed and marketed. In 1902 there was but one machine on the market; at the Convention in 1905 there were 21 block machines on exhibition. These blocks were expected to revolutionize building because their durability and resistance to fire was expected to secure for the building in which they were used, so material a reduction in the fire insurance that they would largely replace wood and other building materials.

A competitive market with a serious lack of standard practice has resulted in conditions so very unsatisfactory for the industry that by the summer of 1904, Mr. Charles C. Brown the Editor of *Municipal Engineering,* at the suggestion of Mr. A. S. J. Gammon of Norfolk, Va., who was interested in making blocks, undertook the organization of the first Convention of 1905. An editorial in the September issue of *Municipal Engineering* gave publicity to the idea and requested the opinion of those interested as to the advisability of forming such an association. There was a hearty response that assured the success of the movement. In the week of October 3, 1904, during the Engineering Congress at the Louisiana Purchase Exposition, St. Louis, an informal meeting was held by those interested.

The original suggestion was to form an association by the manufacturers of cement block machines to educate the users of such machines as to the proper methods of making blocks to insure good results.

After the informal meeting in St. Louis the scope of the organization was extended to cover the various uses of cement for the purpose of bringing about a better knowledge of the art.

The convention was held in the Claypool Hotel, Indianapolis. Ind., January 17-19, 1905; the attendance was about 600 and 47 firms exhibited their products.

The speaker well remembers, while at the St. Louis Exposition, where he was at that time in charge of the Collective Port-

land Cement Exhibit and Model Testing Laboratory, his reluctance in accepting the invitation extended to him by Mr. Brown to address the convention on the uses of cement, because of his impression that it was to be a trade meeting. He was not surprised on arriving at the convention to find that those present were largely interested in the manufacture and use of cement building blocks and the materials and machinery required.

At its first meeting the convention decided to form an association and appropriate committees were appointed for effecting a permanent organization. The Constitution adopted provided that the name of the organization should be the "National Association of Cement Users," and that its objects

> "shall be to disseminate information and experience upon, and to promote the best methods to be employed in the various uses of cement, by means of conventions, the reading and discussion of papers upon materials of a cement nature and their uses, by social and friendly intercourse at such conventions, the exhibition and study of materials, machinery and methods, and to circulate among its members by means of publications the information thus obtained."

The Constitution also provides that,—

> "a candidate for membership must be a company or person engaged in the construction or maintenance of work in which cement is used; a company or person who is qualified by his business relations or practical experience to cooperate in the purposes of this Society, though not himself engaged in the use of cement, and may be a manufacturer of, or a dealer in machinery or supplies for cement users; or a man who has attained eminence in the field of engineering, architecture or applied science, who is interested in cement and its uses."

In the nomination of officers there was trouble; there were two groups of block manufacturers, one interested in "The Palmer Patent," and another interested in various other patents. The speaker has a vivid recollection of answering the telephone in his room in the Claypool Hotel at 3 o'clock in the morning and being requested to come at once to the Columbia Club on a vital matter.

On arriving at the Club he was ushered into a private room thick with smoke, filled with a lot of weary but intensely alert men. He was told that they had agreed upon his nomination for the presidency but wished his assurance that he would adopt the policy they had agreed upon. You can doubtless imagine the speaker's forcible reply and his opinion of men who would pull him from his bed at that hour to offer him something with strings to it. He flatly refused to accept. Later in the morning it was again proffered unconditionally. He remembers his deliberating the matter of acceptance and concluding there were great possibilities in such an organization.

The presiding officer at the opening meeting was John P. Given, of Circleville, Ohio, a representative of the opponents of the Palmer patent; the opposition of the advocates of the Palmer patent resulted in the selection of an engineer as President; a tacit recognition of the neutrality of this class of professional men. The first Board was largely composed of either block makers or block machinery manufacturers. The papers at the first convention—

"Coloring of Concrete"
"Testing of Cement Blocks"
"The Dry Mixture of Concrete"
"Practical Work of Constructing Sidewalks"
"The Water Proofing of Concrete Blocks"
"Waterproofing Concrete"
"Cement Posts"
"Mortar Sand"

were largely those devoted to the manufacture and use of cement blocks.

The Standing Committees during the first five years had assigned to them,—

"Concrete Blocks and Cement Products"
"Streets, Sidewalks and Floors"
"Reinforced-Concrete."
"Art and Architecture"
"Testing Cement and Cement Products"
"Machinery for Cement Users"
"Roadways"
"Fireproofing and Insurance"

"Building Laws and Ordinances"
"Exterior Treatment of Concrete Surfaces"
"Specifications for Cement Products"

Among the first standards adopted were those for

Portland Cement Sidewalks
Cement Hollow Building Blocks, and
Building Regulations.

The impetus resulting from the steps taken to organize the National Association of Cement Users led to the formation of the Northwestern Cement Users' Association, the Iowa Association of Cement Users and the Nebraska Association of Cement Users.

The major topic at the early conventions was the cement block. The speaker from the start held the vision of an organization that would be to the cement industry what the British Iron and Steel Institute was to the steel industry. But progress in those earlier years was slow and at times difficult; there was much opposition and a decided reluctance on the part of Engineers and Architects to join the Association since the use of cement for building purposes was largely confined to pavements and to mass and foundation work in which they were but little interested.

The advocates of cement blocks were believers in dry mixtures for the reason that in the then state of the art a dry mixture was believed to be necessary in making cement blocks. In order to cheapen the product mixtures as lean as 1 to 8 were used which enabled the maker to sell them at an attractive price.

It was then the practice to mix concrete to the consistency of damp earth so that after considerable vigorous tamping the water would just flush to the surface of the concrete.

The speaker well remembers that his vigorous advocacy of a wet concrete developed such an opposition at the Milwaukee Convention that a delegation waited upon him to inform him that unless he agreed to cease his advocacy of wet concrete he would not be supported as a candidate for President. The speaker did not cease his campaign of education nor was he defeated.

In his capacity as President the speaker broadened the program each year and other topics than cement blocks were discussed, such as matters of design, details of construction, and the development of standards.

A review of the publications of the organization will show the transition in the character of the papers presented.

Gradually the character of the work of the Association changed and it became apparent to the speaker that the name was no longer indicative of its work and he accordingly recommended the change of name to the American Concrete Institute, which change was made July 2, 1913.

It should be borne in mind that at the time of the formation of the National Association of Cement Users, those interested in the use of cement—other than for mass and foundation work—were the small users in sidewalk construction and building blocks. The latter branch of the industry was then enjoying a boom as a result of the activities of the block machinery manufacturers, and many were engaging in the manufacture of blocks who knew little of the material. It was because of this need, and to promote uniformity in the blockmaking art, that the first Convention was held.

As time went on these members gradually learned proper methods, and their interest lagged and they discontinued their membership. In their places came members who for the most part were interested in the industry as a whole.

Fortunately this broadened activity for the Institute was its salvation. The cement block made of a dry and lean mixture proved unsatisfactory and the block industry experienced a period of great depression during which the block fell into disfavor.

In those days the Association was held together by the devotion and determination of a small band of pioneers whose unselfishness and faith in the future of the organization kept it alive until the record of its accomplishments and the recognition of its work created a new interest, and its membership under the very intelligent and determined efforts of its officers began to grow; it has recovered the losses due to the depression in the block industry and it is now enjoying that healthy increase in membership which is characteristic of a live organization; its future is assured.

The great opportunity the Annual meetings of the Institute afford for fraternization and exchange of information among those interested in the industry, cannot be over-estimated. It has done much to stimulate interest in the industry, and has thereby greatly stimulated its growth.

The goodfellowship that developed in the earlier days brought men together year after year and made the annual conventions reunions. Another fact which has somewhat retarded growth in membership was that the relatively new industry of reinforced-concrete construction attracted young men who could not afford to spend the money required in attending a Convention; as time went on these men became more prosperous and began to attend. Architects and Engineers interested in other structural materials looked askance at this new material; their technical interest was in older organizations and they could not, therefore, readily become interested in a new organization. As the work of the Institute became more widely recognized men in the industry found it to their interest to attend the Annual Conventions because of the valuable information that could be obtained in the interchange of views as to the best methods of procedure.

The Cement Show was a tremendous power for good; its educational work and stimulation of inventive genius leading to improvements in the art, cannot be adequately appraised.

It was a great burden courageously assumed by the officers of the association and carried successfully until the Cement Products Exhibition Company organized as a commercial enterprise and relieved it of part of the burden. The Association arranged its conventions coincident with the Show. There came a time when the need for such exhibitions was passing; the Exhibition Company decided to discontinue them and none have been held since 1916. The present exhibit is the first the Institute has held since 1909. Although the present exhibit is of a different character, being largely a historical exhibit, it is intended to be educational and an exposition of the development of the Art.

The value of the exhibitions in connection with these conventions should not be lost sight of. They undoubtedly did much to popularize the use of cement and greatly stimulated its consumption.

The amount of membership dues of the Institute of $5.00 per annum was never sufficient for its needs; and in addition to raising the dues to $10.00 per year, it became necessary to raise the very considerable sum of $12,000. This the speaker undertook in 1912 and was successful in raising a fund of more than $18,000 which was the means, under the wise and conservative guidance

of its officers who have devotedly, loyally and unselfishly administered its affairs, of placing the Institute in its present healthy and prosperous condition.

The speaker cannot refrain from availing himself of this opportunity to express his personal thanks for and appreciation of the splendid and substantial response to his appeal for funds for the support of the Institute in the most critical period of its existence. He has the greatest admiration for the men whose broad vision led them to come to the rescue, and he is very grateful that he was able thus to obtain the funds necessary for caring for the deficit and providing for the future, so that at the close of his ten years of service as President he could turn over to his successor a going concern, in sound financial condition.

The dangers of commercialism which beset the infant organization during its earlier years were safely passed, and in its twentieth year it faces the future with an established, honorable position and a prestige resulting from a worth while past.

During this trying period many were the doubting Thomases whose gloomy belief was that the days of usefulness of the institute were over.

Our British sister Institute went through a similar period but met the situation there by a change in the constitution and, in name from "The Concrete Institute" to "The Institution of Structural Engineers" so as to include in the membership structural steel engineers.

The growth of the Institute has kept pace with the development of the industry. At the time of its formation the production of American portland cement was about one million barrels; it is now over hundred and thirty-seven million barrels, an increase of 137 times that sum.

The attitude of the public is well illustrated by the cement sidewalk. The speaker well remembers the earlier days when sidewalks were regarded as undesirable and many were the objections to them. A badly laid sidewalk that cracked was immediately cited as evidence of its unsuitableness as contrasted with flagstone, brick or even wood. But the intrinsic value of cement sidewalks, as regards first cost, upkeep and smoothness of surface, and useability in all weather, has overcome captious criticism.

So it has been for the various new uses for concrete.

The use of cement in the manufacture of artificial ornamental stone antedated the birth of the Institute.

The art has advanced greatly since that time and artificial stone is now produced having a texture superior to that of natural stone and for much less money. When the carving of natural stone is considered it is found that artificial stone cast in the most intricate design costs materially less money.

While it was used in the early eighties for sidewalks, no one at that time had attempted to use concrete for roads. The speaker inspected roads laid in Vienna in 1906. The first concrete road in the United States so far as the speaker is informed was built in 1907.

The concrete road has gradually demonstrated its undeniable superiority over other road materials and is now recognized as the premier material.

The First Concrete Road Conference was held in Chicago in February 1914; at this conference there was manifested a widespread desire for information; at the Second Conference held in Chicago in February 1916, it was evident that the use of portland cement for roads was well established and that the problem was only a matter of details of construction.

The consumption of Portland Cement for roads and highways has attained a total of 28,000,000 barrels of which more than 18,000,000 are used for roads.

The reinforced-concrete pavilion in Spanish mission style, which was the main feature of the Collective Portland Cement Exhibit in the St. Louis Exposition in 1904 was regarded as a curiosity, a new thing, although reinforced-concrete buildings had been erected for some time in this country.

Reinforced-concrete had advanced from a largely experimental stage to that of a preferred material for buildings not only on account of its low cost of construction and maintenance but also because of its resistance to fire and other destructive agencies. There was an early belief that it was a seasonal material and could not be used in cold weather. The art has so advanced that it is now used all the year round.

A building of concrete formerly required a much longer time to erect than did a similar one of steel; but now it can be erected with as great and generally greater speed than the steel building.

Many will recall the building code fights especially in New York where producers of competitive materials, sensing the growth of a dangerous rival, endeavored to have requirements written in to building codes to handicap this material and practically prevent its rise. The restrictions were so great that concrete building construction was not possible in Manhattan and but few structures were erected in the territory outside of Greater New York. The pioneer work done by Henry C. Turner, Leonard C. Wason, A. L. Johnson, Roos F. Tucker and others in courageously and persistently working to secure contracts for reinforced-concrete buildings and in securing better code provisions, has borne fruit beyond belief.

The construction of bridges afforded an excellent field for the use of concrete; it was found that durable, attractive, artistic, economical structures could be constructed of reinforced-concrete and gradually concrete bridges began to be built. At first they were a novelty but as they increased in number and length of span, they were accepted as a matter of course.

Fence posts were considered at the first convention but the few that were made were costly, unsatisfactory and not particularly durable. The art has now advanced to where they are not only in general use but their proved economy as to cost of maintenance and the superior character of the resulting fence renders them preferable where durability and low maintenance are considerations.

Another forward step in the development of concrete was its introduction on the farm. The concrete silo besides being durable and fire resistive was found to keep the ensilage in better condition than the ordinary silo and its use for this purpose became popular among farmers.

The feeding floor for stock was also found economical as regards the feed and the health of the stock. Then followed its use for sanitary cow barns, chicken houses, drain tile, etc., in all of which uses concrete proved to be both cheap and durable.

An early use of concrete was in connection with sewers; such use was, however, confined to pipes of the smaller sizes. As the knowledge of reinforced-concrete increased, the size of sewers increased, and it is now found that reinforced-concrete is the most economic material for such purposes. It is of demonstrated value

for ordinary pipes and conduits, and for high-pressure pipes it is entering a new and increasing field. Cement products are meeting with favor both as to durability and low cost. Cement is being used in the manufacture of telephone poles, and battery cells; it is in demand for use in tanks, reservoirs, dams, walls and abutments and in fact for all purposes where structural materials are required.

The criticism of the behavior of cement in sea water has been general in the last twenty years and efforts have been made to inaugurate an elaborate investigation of the subject. It has always seemed to the speaker that such critics lose sight of one very important factor, viz., that, of all the structural materials used in sea water, concrete has the best record. Generally the so-called failures of concrete have resulted from its improper use and from bad workmanship. There are many examples of structures of concrete that have been erected in sea water and are still in excellent condition. Such failures as are reported occur in the structure between the range of the tides—there have been few if any failures that have originated below low water. Certainly it is a fact that during the last twenty years an increasing number of concrete structures have been built in sea water.

During the war the great need for ships led to the use of reinforced-concrete for this purpose; but for demonstration of the value of the concrete ship, the war terminated too quickly. It was hardly to be expected that such a new and special application could be wholly successful at its first trial. There was the strong opposition of the users of competitive materials, and the prejudice of shipping men. While recognizing that a battleship built of steel many times heavier for equal bulk than concrete, floated and was sea worthy, nevertheless they could not recognize concrete, an artificial stone, as capable of floating.

In the opinion of the speaker the time will come when reinforced-concrete will be used for shipbuilding and will prove economical as to first cost, durability, and cost of maintenance. What other material could have withstood so successfully the merciless pounding of the waves as has the Pollias which was driven on the rocks off the coast of Maine more than five years ago and is still intact.

An important factor which should not be overlooked in the analysis of the development of the concrete industry is the stimulation of that industry during the period of the war.

The production demanded to meet the requirement of the overseas forces was so great that restrictive orders were necessary to curtail building to that which was essential for winning the war, or to meeting civilian necessities.

The requirements for pig iron in the manufacture of munitions of war were far beyond the productive capacity available, so that it was necessary to use other material than steel for building wherever possible. Reinforced-concrete construction was required by the Building Materials Division of the War Industries Board, and this of necessity led to it use by those accustomed to the use of structural steel, and also for purposes for which it had not been generally used heretofore.

The use under these conditions of duress compelled a trial of the material which afforded an opportunity for demonstrating its intrinsic merits as to economy of construction, durability, fire resistance and economy of maintenance. Where the material was thus used there generally has been no desire to return to other materials and this has been a factor in increasing the use of cement.

The war also gave a tremendous impetus to the use of the motor truck. The inability of the railroads to handle the traffic made the use of the motor truck imperative; and this term necessitated suitable hard roads which led to a further and widespread use of concrete. Wherever this material has been used for such purposes the satisfaction of the user is so great that the general demand for these roads is far in excess of ability to supply them.

Concrete continues to be one of the most satisfactory of the materials available for fire resistive construction. Since the Concrete Institute was founded a number of notable events have tested the properties of concrete in its resistance to fire and earthquake. The San Francisco fire following a disastrous earthquake afforded some measure of the value of the resistive properties of Concrete. although the number of concrete structures was not large. But such buildings as existed satisfactorily withstood both the fire and earthquake. Although it is perhaps too soon to draw conclusions (because all the facts are as yet not available) concerning the recent Japanese earthquake and fire, yet it appears from present

facts that a relatively poor concrete stood the earthquake and fire test in a very satisfactory way. Concrete, as a result of this test, is established more firmly than ever as a suitable material for such purposes. The Edison fire was perhaps the most outstanding test of the fire resistive properties of concrete. No similar test is recorded. It is the consensus of opinion of those who investigated the condition of the concrete of the buildings of the Edison plant after the fire that its behavior was wholly meritorious. The failure of other building materials under the test of that fire was in striking contrast to the resistance of the concrete.

Two other fires that may be mentioned as contributing to our knowledge of concrete are the Far Rockaway Fire and that in the plant of the Barrett Company in Philadelphia. In both of these fires it was evident that serious damage was done to concrete composed of quartz aggregate under the condition of a very intense rapidly developed fire.

These tests confirming the statements of Prof. Ira H. Woolson based upon the results of his experiments made under the direction of the Sub-Committee on Tests of the Joint Committee on Concrete and Reinforced-Concrete as reported in the Proceedings of the American Society for Testing Materials (Vol. V, 1905, Vol. VI, 1906 and Vol. VII, 1907) and the later investigations presented in his report on the Far Rockaway Fire. Subsequent investigations of Thames Ballast by the British Fire Prevention Committee and the still later investigations by the Bureau of Standards confirm these conclusions.

The work that has been done since the founding of the Institute both in research in the laboratory and the intensive study by Committees has advanced the knowledge of the properties of both concrete and reinforced-concrete in matters of design and application.

The educational value of the report of the first Joint Committee on Concrete and Reinforced-Concrete can hardly be estimated. Its recommendations went a long way toward establishing confidence in this material and as its use began to grow its demonstrated intrinsic merits became apparent and still further enhanced this confidence.

In time it became evident that these recommended practices should be crystallized in definite specifications for Concrete and

Reinforced-Concrete for use in contracts and also as the basis for building codes, thereby still further standardizing good practice and thus advancing the Art.

The Joint Committee on Standard Specifications for Concrete and Reinforced-Concrete was organized in 1921 and after a year submitted a tentative specification which it is expected will be adopted as Standard during the current year.

Perhaps the most signal advance made, has been in the field practice of mixing and placing concrete; the tentative specifications for concrete and reinforced-concrete with the requirements relating to the quality of concrete, which caused the field investigations that are now in progress, have done more than any other agency to stimulate better field practice.

The speaker is very grateful for the opportunity for service that fell to his lot as Secretary of the Committee on Uniform Methods of Tests of Cement,—as Secretary of the Committee on Standard Specifications for Cement, and also of the Joint Committee on Concrete and Reinforced-Concrete, and as Chairman of the Joint Committee on Standard Specifications for Concrete and Reinforced-Concrete.

The growth of the Cement industry in the last twenty years naturally is a reflection of the use of cement in building and it is probable that this increase has been largely in building and especially in the construction of concrete roads.

The per capita consumption of Portland Cement for the year 1904 as given in the statistical reports of the U. S. Geological Survey was 26,699,350 bbl.; in 1914 it was 84,418,665 bbl., while for last year it is estimated to be 137,000,000 bbl. The increase from 1904 to 1914 was over 200 per cent; and from 1914 to 1923 it was about 60 per cent.

It is interesting to know that early experimenters sought to recover the "so called" lost Roman art of making cement; in the centuries that have elapsed since Rome was the Imperial City, Portland Cement resulting from such researches has come into existence; and the old Roman cement only bears a relationship to natural cement similar to that which natural cement bears to Portland Cement.

The recently created Committee of the American Society of Civil Engineers on Cement and the cooperative researches about

to be inaugurated by the United States Bureau of Standards and the Portland Cement Association will turn the scientific mind more particularly to the constitution of Cement and farther improvement in its quality may be made.

The increase in the production of Portland Cement has been little short of marvelous. In the early nineties when the speaker began his career in the Cement industry, only a limited quantity of American Portland Cement was produced; a very large quantity was imported from Germany and other foreign countries; the production in the United States was only about 25 per cent of the world's production. Today that percentage has so grown that the production of this country is greater than the combined production by all the other countries; and the maximum of American production is not yet in sight.

CHAIRMAN HUMPHREY.—*During this remarkable period of progress there has been a prominent figure—a man well informed on the industry—and engineer whose devotion to it is second to none—a most active and helpful member of a number of Technical Committees—for a number of years Chairman of its Technical Problems' Committee and now the honored Chief Executive of the Portland Cement Association.*

A man whom the Institute is fortunate in having as a speaker and who is peculiarly qualified to address you on

"Cement the Magic of Concrete."

It is my privilege and pleasure to present

MR. F. W. KELLEY.

CEMENT IS THE MAGIC OF CONCRETE.

BY F. W. KELLEY

PRESIDENT, PORTLAND CEMENT ASSOCIATION.

THE efforts of man to find a binding material for use in building, started with the prehistoric Better Homes movement, when primitive man moved from the most desirable cave to the detached villa of mud or of stone. Imposing monuments to his deity also needed a binder.

Large deposits of silicious clays containing alumina are widely distributed throughout the world, while lime exists in some form wherever the seas have covered. Man found, probably by accident, that one, or better still, both of these materials were desirable chemical elements in a satisfactory binder.

In the production of these binders heat has been used, first in the early sun dried muds, later in the moderate temperatures needed for burning lime and for natural cement, and finally in the white heat required for making portland cement.

In the use of all these binders or cements, water has been essential.

It is a long and interesting history upon which we can only touch lightly.

The Egyptians used as mortar in the great pyramid, gypsum binder made by lightly burning the crude gypsum rock, of which

the blocks in the pyramid are composed. This would be satisfactory only in a climate like that of ancient Egypt.

The Romans added to powdered lime a silicious material of volcanic origin, and gave to the mixture the name of puzzolanic cement.

The English burned and ground what were known as "Septaria Nodules," containing lime and silica, dredged in certain localities, and called it "Roman" cement.

Natural cements were produced in many countries, from natural rock, which experience showed was of suitable composition, by lightly burning it to drive off the combined carbon dioxide, and grinding it to a coarse powder.

It was not until Smeaton's investigation about 1756, in connection with the re-building of the Eddystone Light House, that the records showed some knowledge of chemistry, and of the reason for the property of hardening both in air and in water possessed by the so-called hydraulic cements containing both lime and silica with certain other materials.

In October, 1824, nearly one hundred years ago, Joseph Aspdin, an English bricklayer, took out a patent on what he called portland cement. As is well known, the name portland was given because of the similarity of this cement when set to the building stone quarried on the Isle of Portland, England. Aspdin did not understand the reason for the results which he obtained, and it remained for Isaac Johnson, also an Englishman, to develop and perfect methods by which a uniformly reliable product could be obtained.

The definite control of the raw mix, and the extreme high temperature of burning, which distinguish portland cement manufacture, were now brought into practice for the first time.

The production of portland cement developed rapidly in England from about 1850, spreading about the same time to the continent, where it made marked progress in both Germany and France.

In the United States portland cement was imported until about 1872, when its manufacture was begun on a very small scale. A number of attempts were made in widely separated places to get the required results, under the conditions existing in this country.

Portland cement manufacture as then practiced in England and upon the continent, used as raw materials principally soft,

finely divided muds and clays, having the necessary chemical constituents. These materials were moulded into brick, which were burned in vertical kilns, and when cooled, were hand sorted and ground to finished cement.

In the United States there was little or no raw material of the European kind, while the relatively high cost of labor made the European methods of handling too expensive. American raw materials required the development of grinding machinery which could economically reduce hard rock to the extreme fineness required in preparing the raw material for portland cement manufacture, while American economic conditions forced a method of burning which eliminated hand labor. American inventive genius solved the grinding problem, while the burning was accomplished through the use of the rotary kiln. This device, invented in England, was perfected and put into practical use in this country.

The development of the portland cement industry in the United States has been largely due to five factors:

First. The systematic application of chemistry, making possible the use of many different kinds of raw materials, all having the necessary chemical composition.

Second. The development of efficient grinding machinery to reduce hard raw materials to proper condition for the raw mix.

Third. The development of the rotary kiln producing a uniform well-burned clinker with relatively cheap fuel, and eliminating hand labor.

Fourth. The development of standard specifications, establishing minimum requirements for the product; permitting its manufacture from widely different materials, in places reasonably near to centers of consumption, and insuring a reliable cement, at all places, at a reasonable cost.

Fifth. The systematic study of the best uses for concrete, and best methods for making it, thus insuring as far as possible, the most economical and satisfactory results to users

The manufacture of portland cement, as carried on in the United States, is an exact chemical-mechanical process, in which large quantities of raw materials are first combined in the right chemical proportions, and are ground to extreme fineness. The

proportioning of materials is accomplished in machines, under the constant control of chemists. The grinding is generally done in several stages by machines each suited to the particular kind of material handled and to the fineness to be reached.

The raw materials when prepared and standardized chemically and physically are in all cases fed to rotary kilns. These are steel cylinders, rotated slowly about their axes, which are inclined slightly from the horizontal. The raw material fed into the upper end of the kiln, through the action of gravity combined with rotation, slowly works its way toward the lower or discharge end, meeting the hot gases of combustion from the flame of a mixture of powdered coal and air, which is constantly blown through the hood, covering all but a small aperture in the lower end of the kiln, through which the hot clinker is discharged. In some places oil or gas is used instead of pulverized coal.

The effect of the hot gases upon the raw material is first to drive off the moisture and carbon dioxide. Next the materials slightly soften, and roll into balls of a size between that of small bird shot and that of walnuts. These nodules finally reach the white hot zone, where a temperature of about 2800 deg. F. is maintained. At this point a chemical change takes place in the nodules, which up to this time have been a mechanical mixture, and a new material known as portland cement clinker is formed.

The slightly cooled clinker upon issuing from the kiln passes through some form of cooling device and is then ready for the final grinding which reduces it to the fineness of portland cement of commerce.

It should here be noted that in order to secure the temperature necessary for burning, plants using coal must maintain a separate grinding department, in which a large quantity of highly volatile bituminous coal is handled, dried and ground to extreme fineness, so that when blown into the kiln with a suitable proportion of air under pressure, it will at once provide the intense heat required to produce the chemical change, which gives to portland cement its ability to harden and meet the strength requirements of the specification. The quantity of coal or its equivalent needed to produce a barrel of portland cement in most plants approximates half the weight of the cement.

The cooled clinker is generally reduced by two stages of grinding, to the fineness required by the specifications, so that at least 78 per cent of the product will pass through a sieve having 40,000 meshes per square inch. This sieve is finer than most silks, and will hold water. In the final stage of grinding a small quantity of gypsum is introduced with the clinker to insure a setting time which will make the resulting concrete convenient to handle.

MODEL OF TYPICAL CEMENT PLANT.

The finished cement is carefully sampled and tested, to insure full compliance with the specification requirements, and is held in bulk in large storage bins until placed in weighed packages for shipment. In most plants very large storage capacity is provided to care for variation in demand due to seasonal or other causes.

There are over eighty distinct operations required in many plants to produce portland cement, although it should be understood that scarcely two cement plants are alike since each has

been designed to take its particular raw materials and by means which are as diverse as the materials, reach a substantially uniform end.

Rock used for making portland cement is drilled by power drills and shattered by explosives. The rock and other raw materials are generally loaded by steam or electric shovels on cars and conveyed to the crushers. Coal is handled and stored by cranes or mechanical conveyors of some type.

Huge crushers built on the principle of a nut cracker and exerting an enormous pressure accomplish the reduction of rock or other hard material, while machines operating on the principle of pounding and rubbing, pulverize the several materials through the remaining stages to their final condition of impalpable powder. The initial crushing machines are generally slow speed, while the pulverizing machines may be high speed, when of the mortar and pestle type, or slow speed when of the ball and tube mill type. Similar types of machines are used for similar reductions in size, of raw materials, of coal, and of finished cement.

The method of handling the raw materials with water differentiates the wet process from the dry process of making cement. Coal and clinker grinding operations are the same in both processes.

The rotary kiln, which has been described, sometimes reaches a size equal in diameter to a standard Pullman car, and in length equal to three such cars, placed end to end. Some of the grinding machines weigh, with their load of grinding elements, as much as a large locomotive. The mechanical conveying devices used for transporting the materials, horizontally and vertically include belt, pan, and screw conveyors, bucket and pan elevators, cranes, hoists, cableways, drags, skips, cars and compressed air.

The cement when forwarded to the user is packed and weighed in pre-tied bags of special construction and is only touched by hand when placed in the freight car, truck or boat for shipment.

Large amounts of power are required for the several grinding operations. The portland cement industry is one of the three industries of the United States using most power per dollar value of product. In many cement plants the hot gases issuing from the kiln are used to make steam for power.

A well-equipped machine shop is an essential part of every cement plant, since the abrasive character of the raw materials, and of the finished product, which is capable of scratching glass, causes a very high rate of wear, and involves large repair costs.

The capital investment in plant is high, the capital turnover is slow, and manufacturers have had to rely upon a large volume of business to make profits on this commodity, which sells at a pound price lower than any other highly manufactured article.

In 1890 there were sixteen plants in the United States producing a total of 335,500 bbls. per annum, or an average per plant of about 21,000 bbls. per annum. In 1900 the number of plants had increased to fifty, the total production to 8,482,000 bbls. and the average plant production to 175,000 bbls. per annum. In 1903, just before the establishment of the American Concrete Institute, the number of plants had increased to seventy-eight, located in 19 states, the total production being 22,342,973 bbls. and the average plant production 285,000 bbls. per annum. In 1922 the number of plants was one hundred and eighteen, in 27 states; the production 114,789,984 bbls., and the average per plant, 972,000 bbls. per annum.

In 1923 the total production was about 137,000,000 bbls. or 375,000 bbls. per day. This daily output is greater than the annual production in 1890.

The average annual output per rotary kiln in 1922, which is the last year for which official records are available, was 151,000 bbls. compared with 85,000 per kiln in 1910.

It is estimated that the cement plants in the United States used in 1923 over 10,500,000 tons of coal, 4,700,000 bbls. of fuel oil and 4,000,000,000 cu. ft. of gas.

There are probably over 225,000,000 cloth sacks in service handling cement today. Over 60,000,000 sacks are lost or destroyed per annum. A strip of cloth 34,000 miles long and 30 in. wide is needed to replace them. Over 43,000,000 paper sacks were used during the year. Over 38,000,000 bbls. of lubricants were used.

For the kiln linings over 5,400,000 fire brick were needed, over 2,000,000 lin. ft. of belting were worn out, over 16,000,000 lbs. of explosives were used, and over 725,000 tons of gypsum were consumed.

The consumption of cement in the United States in 1903 was at the rate of 0.276 bbls. per capita. In 1923 this consumption was about 1.20 bbls. per capita. In 1922 there were seven states in which the per capita consumption averaged 1.66 bbls. per person. The consumption in the country in 1923 was at the rate of about 450,000 barrels per working day, 45,000 bbls. per working hour, 750 bbls. or five carloads per working minute and twelve barrels per working second.

In 1923 the cement industry accounted for about 3 per cent of the tonnage moved by the railroads of the United States.

In 1903 the world's production of portland cement was estimated at about 66,000,000 bbls., of which the United States produced about one-third.

In 1923 the United States produced about one-half the estimated production of the world as follows:

United States	137,000,000 bbl.
Germany and Austria	30,000,000 bbl
British Empire	35,000,000 bbl.
France and Colonies	12,000,000 bbl.
Japanese Empire	12,000,000 bbl.
Belgium	10,000,000 bbl.
Others	30,000,000 bbl.
Total	266,000,000 bbl.

The quality of portland cement manufactured in the United States has constantly improved. The extreme 200-mesh fineness of today noted above, has superseded the coarser product which thirty years ago was measured only by 50- and 100-mesh sieves, and even 20 years ago the 200-mesh sieve was just about to be recognized.

The present strength requirements of 200 lb. at 7 days and 300 lbs. at 28 days for 1:3 sand tensile briquettes, are not only much higher than formerly, but are measured by methods which make the real increase in strength requirements at least 10 per cent more than shown by the figures.

Thirty years ago 1:3 mortar strength of 65 to 80 lb. at 7 days and 100 to 200 lb. per sq. in. at 28 days were considered reasonable. Twenty years ago 150 lb. at 7 days and 220 lb. at 28 days was the requirement of the U. S. Army Engineers.

The requirements which make up the specification, were added from time to time, as experience showed that a cement which met the requirement gave good results in service. More recently most of the specification requirements have been checked by careful laboratory research. A cement which meets the specification requirements will give good results if used right.

There has been also a marked improvement in the care and thoroughness of manufacture, and in the resulting uniformity and reliability of product. A requirement for membership in the Portland Cement Association is that members' product must be guaranteed to meet the standard specification.

Portland cement as manufactured in the United States is the result of the accumulated experience of a century, refined by modern scientific methods of control and manufacture, and tested by time.

The portland cement manufacturer deals with elemental forces. The force required to reduce the rock to a condition approaching its elementary fineness, must overcome the enormous earth pressure which compacted these rocks in the past geologic ages.

The temperature of burning, which is among the highest known in large scale manufacturing operations, approaches the point where matter ceases to retain the forms with which we are familiar, and flashes forth as the flaming gases we see in the stellar cosmos.

Quite appropriately out of this play of elemental forces there emerges a product—portland cement—which gives to man a creative power.

In itself cement is only potentially useful. It cannot practically be separated from its use. It cannot use itself. Man must use it, and anybody anywhere may attempt to use it.

This fact has been a determinating one in the work of the cement manufacturers of the United States, who maintain the Portland Cement Association, an engineering service organization, national in extent, with 28 district offices placed near centers of consumption. Through its Structural Materials Research Laboratory and its field men, it is in a position to know the best practices in concrete construction, and it gives freely advice and assistance to all who ask it.

Any man in this country can without expense learn how to use cement right. Careful attention to details and personal inspection are needed to insure that the right methods are followed in concrete work. The use of cement is both a science and an art.

Cement has been called the magic powder or the magic of concrete, and like the Jinn or Genii of the Arabian Nights tales, it requires careful control by its master if it is to behave properly. The cement manufacturer captures the Genii and locks him in the powder made from portland cement clinker, handing the key to the cement user in the form of water. He who uses this key assumes a responsibility which cannot be shirked. Instantly a tremendous force is unlocked. The everlasting grip of the Genii begins to envelop the materials nearest at hand, and for all time there is reflected in the resulting concrete the knowledge, care and thoroughness of the user, in these first few hours after the force is released The user must supply proper aggregates and water, and use proper methods for mixing, placing and curing. He must help the Genii get the right kind of grip upon the right materials.

The quality of the concrete produced has a significance beyond that of the structure of which it is a part—it is a test of the man who created it.

The American Concrete Institute grew out of the natural desire of cement users to compare experiences in the use of cement, and to profit from them. The Institute can be a most important factor in promoting the right use of cement and concrete.

The continued expansion and development of the cement industry in this country depends upon the general adoption of the methods for making concrete.

CHAIRMAN HUMPHREY.—*When in Rome it was my good fortune to travel over some of the roads that had made that City famous—in jolting over the Appian Way—that an Englishman in the party thought was not the 'appier way"—the speaker was reminded of travel on some of the roads in this country before the advent of the concrete road, in the early days of this Institute. Before and through this period, it has been my pleasure to know the next speaker, whose investigations and researches on the subject give him the right to address you with authority and with a knowledge of the history of*

"Concrete Roads"

and the speaker esteems the opportunity he has in presenting to you

PROF. A. N. JOHNSON.

THE USE OF CONCRETE IN ROAD CONSTRUCTION.

BY A. N. JOHNSON

DEAN, COLLEGE OF ENGINEERING, UNIVERSITY OF MARYLAND

When the author was invited by the committee of the American Concrete Institute, having in charge the program of the 20th Anniversary Meeting, to prepare a paper on the use of concrete in road construction, he accepted with the proviso that there should be assigned someone to gather historical references and other data which such a paper should properly contain. Further, the author suggested that he would prefer to have Robert Kingery, a former colleague, to assist him. Through the courtesy of William M. Kinney, Manager of the Portland Cement Association, at the request of the Program Committee of the Institute, Mr. Kingery was assigned to this task. How well and thoroughly he has performed his work, the mass of data and the bibliography which are contained in this paper bears striking evidence. And I desire here to express the deep obligation I am under personally for the assistance so generously given me by Mr. Kingery.

INTRODUCTORY.

It has been the purpose in the compilation of the historical material for this topic to confine it to the period coincident with the development of modern road construction, following the increased, need for transportation by land that began to make its influence felt about the middle of the seventeenth century, but which

GLENGYLE TERRACE, EDINBURGH, SCOTLAND.
A 50-YEAR-OLD CONCRETE STREET.

did not materialize to any considerable extent in road building until nearly a century later. It is not amiss, however, briefly to mention some of the references in ancient history to the art of road building.

Whenever a great civilization developed, it depended on transportation, and, to the extent that this was carriage by land, highways were constructed. Remains of some of the ancient roads are still in existence, one of the oldest being a stone causeway, west of the Great Pyramid, which, Herodotus tells us, was built by the Egyptians something over 6,000 years ago, and over which blocks of stone were hauled for the construction of the Pyramids. From Babylon, during the rise of the Assyrian Empire, there radiated a system of highways built about 1900 B. C.* [1]

Coincident with Roman conquest, the necessity arose for means of land communication, primarily as a military measure; and secondly, for an almost equal need for economic development. Thus it was that the building of a great highway system was a fixed policy of the Roman Empire during many years. Excavations which uncovered some of the remains of old Roman roads in England, between Westchester and London, brought to light several sections constructed with a concrete base evidently formed of natural hydraulic cement, mixed with pebbles, the concrete layer being about 12 inches thick. With the decadence of Roman power and civilization, the demands for transportation steadily decreased; the machinery of government was gone; the public works that had been built, especially the highways, were abandoned, and it was no one's care to keep them in repair.

The spirit aroused by religious enthusiasm in the twelfth century revived but a faint demand for means of transportation by land. There resulted numerous instances of repairs to highways and bridges undertaken by a religious or semi-religious confraternity known as the "Brothers of the Bridge" (Fratres Pontis or Fratres Pontifices), who seem to have been active over a period of one or two centuries.

About this period, the Incas constructed in Peru some really remarkable examples of roads, which, however, were not adapted for vehicles of any size. Soon after the occupation by the Span-

---

*These figures are references to be found at the end of this paper.

iards, under Pizarro, in the sixteenth century, these roads fell into disuse.[2 3]

We now will pass to that period which marks the beginning of modern highway construction.

EARLIER USES OF CONCRETE FOR ROADS.

In the latter part of the eighteenth century began the construction of the highway system of France, which was developed, to a large degree, by the French engineer, Tresaguet. This remarkable highway system was brought nearly to its present extent under Napoleon.

There is, however, no evidence that concrete played any part in this earlier construction. There occurs, however, a very interesting comment by Bry. Higgins, M.D., in his book, "Experiments and Observations on Carcareous Cements," London 1780, in which he says (p. 232), "I have thought that the small stones, which constitute the gravel chosen for our roads, could not be reduced to dust so soon as they now are, by the heavy carriages, if they were firmly imbedded in a small quantity of coarse and good calcareous cement, so that the bodies which roll over them should rather compress them, than grind them against each other as they do at present. And as the frequent failures of pavement are manifestly owing to the infirmness of the ground and the looseness of the stones, I have imagined that a solid bed of cementitious work, in the manner of the Romans, and the settling of the paving stones in good mortar, would ultimately lessen rather than enhance the expense. I offer these conjectures in the hope that nobody will presume to decide on the subject, who does not know the difference between the common mortar, and the best that can be made of lime and sand; and that some public-spirited man will make the experiment, where lime is cheap and the expense of pavement or of gravel is considerable. If the expense should be found too great for any public works of this kind, the same measures may nevertheless be tried in private areas and walks, in which the neatness, duration, and prevention of vegetation, may compensate for the extraordinary price."

It does not appear that this suggestion was acted upon until 1838, when the first use of concrete in our modern roads is found

in the construction of the highway from London to Holyhead, which, with packets from this latter point, joined London with Ireland. Credit for the consummation of this project is due, in no small degree, to the efforts of Sir Henry Parnell who secured, through Parliament, the necessary grants for its construction. In 1828 he published a treatise on roads in which is found a very interesting and detailed account of this work.[4]

In particular, there is included in this book the sixth annual report of Thomas Telford, who was charged with the construction of this road. Our interest centers in the account of the section, about one-quarter mile in length, known as the Highgate Archway Road in Hornsey, a portion of London, which describes the laying of a concrete base. This particular section of the road had given much trouble; the ordinary construction followed by Telford, and known today under his name, having proved ineffective, it was determined to re-lay this, using a concrete base. This was laid between June and September in 1828, and details of the construction are described in testimony given by Sir John McNeill (who was the resident engineer in charge) before a Parliamentary Committee of inquiry into methods of road construction. The concrete, he states, was made of Parker's Roman cement, which was patented in England as early as 1796. This cement was mixed with eight times as much washed gravel and sand, and was delivered at a cost of two shillings per bushel. The pavement was built six yards wide and six inches thick. "The cement was laid on, mixing it first in a box with water, gravel, and sand, in certain proportions." A triangular piece of wood, sheeted with iron, was pressed into the concrete at 4-in. intervals. This was to provide a bond for a layer of broken stone, of which the surface was built.

It is reported the maintenance of this road was 900£ per year. It is not clear, however, just how long a section of the road this included, but the report states that with the concrete base the maintenance was very much less.

There are many other interesting features brought out in this report, among them mention of a method of measuring the tractive effort by a specially constructed wagon devised by Sir John (noted later under the topic of research), who reports that while the stone road constructed in the ordinary manner required 156 lb. to draw this wagon, but 56 lb. were required over the section constructed

with the concrete base; also, that while a portion of the old road was worn 4 in. in depth between October and March, less than 1 in. was worn away on the section with the concrete base.

No further reference to the use of concrete in road construction is found until thirty years later, when there was laid on three sides of the Palais Royal in Paris an asphalt pavement, which had a 6-in. concrete base.[5]

Eleven years later, in May, 1869, there was laid on Threadneedle Street near Fitch Lane, in London, an asphalt pavement on an 8-in. concrete base, and, in October of the following year, another asphalt pavement was laid in Cheapside and the Poultry on a 9-in. concrete base.[6]

In all of the instances so far recorded, it appears that concrete was used in road construction as a base or foundation course, but had not been used for the wearing or surface layer.

In the proceedings of the Edinburgh & Leigh Engineers' Society, 1874-75, there is a paper on "Streets" by J. H. Cunningham, in which is described the first construction where the concrete served as the wearing surface. This road was laid in 1865 at Inverness, Scotland, "on the approach to the goods station of the railway." Mr. Cunningham states in his paper that in 1870, after the road had been under traffic for four and one-half years, the wear of the surface was scarcely appreciable.

He describes also another section of concrete road about 50 yd. long and 15 yd. wide, laid in 1866 on the George IV Bridge, Edinburgh. These pieces of concrete street pavements were constructed by Joseph Mitchell and are described by Mr. Cunningham[7] as follows: "The broken metal should be of the hardest quality of uniform size, thoroughly screened, and it should, when in the screens, be dipped up and down in a large tub of water and then thrown on the platform on which the concrete is to be made. Cement of the best quality must be employed, and the sand should be sharp, clean, and gritty. The surface of the ground is brought to form, and rolled several times. The concrete is then laid on the surface in a layer of three or four inches, and is left for three days to harden. The second layer of three or four inches is next laid on the first, and immediately rolled to form with a heavy iron roller, as heavy as two or three men can draw. The cement should be left for three weeks to allow it to become quite hard, before the

road is opened for traffic, although a week has been found to be a sufficient interval."

A third section of concrete paving was built in Edinburgh in 1872 on Gillespie Crescent, and is described by R. L. Bendall, Esq., of Bournemouth, Hantz, England, in an article in *Concrete Highway Magazine,* vol. 4, 1920. "The specifications provided for broken stone foundation about 6 in. thick, bonded and rolled. This was overlaid with a 5-in. layer of Barnton machine broken metal, rolled and grouted with a mixture of one part London portland cement, and one and one-half parts clean Musselburgh sea grit passed through a fine mesh. The work was carried out directly by the staff of the Board Trust." The pavement was still in service in 1920, at the time Mr. Bendall wrote this article.

Two other sections of concrete roads were laid in Scotland in 1873. One is described in *Concrete and Constructional Engineering,* London, April, 1923 (page 270) as follows: "An all concrete road was laid at Blackwood Crescent, Edinburgh, (Scotland) in July, 1873. Mr. J. Sims, the City Road Surveyor, stated in 1920 that the surface of this forty-seven year old road was still in fair condition. A total cost of maintenance was only 40£ since the road was laid. The road is about one-quarter mile long."

The other section was laid in Glengyle Terrace, Edinburgh, according to a reference on the reverse side of a photograph presented to W. F. Long, Salt Lake City, by Wm. B. Grieg, City Engineer, Point Grey Municipality, Vancouver, B. C.

The first piece of "mineralized" wood block (dried in drying chambers and then immersed in tanks of mineral oils) on a concrete base was laid in Grace Church Street, London, near Talbot Court, in August, 1872. On a bed of concrete 4 in. thick, mineralized elm blocks were bedded in portland cement.[8]

The next pavement of this type to be laid in London was placed in 1874, in Fore Street on a 6-in. concrete base, and in 1875 Coleman Street was paved with mineralized fir on a concrete foundation 6 in. thick.[8]

Gannon Street, London[8] was paved in 1874 with fir block on a 9-in. portland cement concrete foundation. At this time there were many pavements of different surfaces being laid in London on a concrete base.

The practice of using concrete as a pavement base now became very general, particularly in the larger European cities, and shortly thereafter in the principal American cities.

EARLIER CONCRETE PAVEMENTS IN AMERICA.

George W. Tillson states that concrete was first regularly used as a base for street pavement in New York in 1888, and that as a pavement it first appeared in Wichita, Kansas. This was a patented pavement known as Jasperite. It was constructed of portland cement and an especially hard rock found in the vicinity of Sioux Falls, S. D. Several thousand yards were laid, but were not a success.[9]

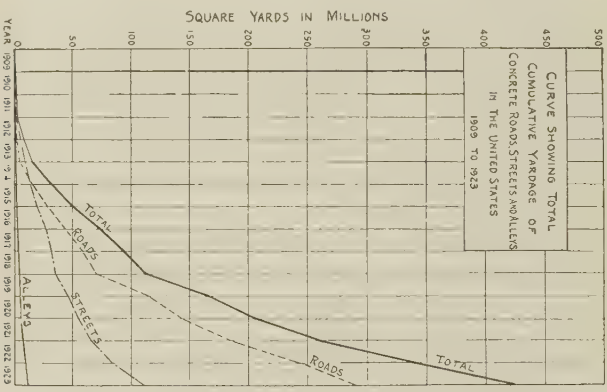


FIG. 1.—CURVE OF CUMULATIVE YARDAGE, 1909-23.

The next instance that we have of the use of concrete as a surface paving material is that laid in Bellefontaine, Ohio, where a strip 10 ft. wide was placed about the Court House in 1892 which is described in a letter from City Engineer, Clair A. Inskeep, under date of Oct. 1, 1919, from which the following account was taken. This pavement was laid in blocks 5 ft. square and some repairs have been necessary along the longitudinal joints. In the construction of this pavement "the bottom 4 in. was of 'one part best portland cement, to four parts clean sharp sand and gravel, in which the proportion of the sand is about one-half that of the gravel' the top 2 in. were composed of equal parts of the same portland cement and coarse sand and gravel sifted to the size of a pea."

Among the earliest concrete streets to be constructed in the United States were several streets and alleys in Richmond, Indiana, which were paved with concrete between 1896 and 1906.[10]

In 1909, Wayne County, Michigan, after a preliminary experiment, laid a piece of concrete road one mile long on Woodward Avenue. This is, perhaps, the first stretch of rural highway in America to be paved with concrete. Each year since this date Wayne County has added to its mileage of concrete highways, until at present there are upwards of three hundred miles so paved.[11] It is possible that Los Angeles County, California, built the earliest piece of concrete rural highway, according to a statement made by Halbert P. Gillette in *Engineering and Contracting* for December, 1922, who mentions three pieces of county road which he thinks were constructed in 1908.

By 1912 numerous sections of highways had been built of concrete in Milwaukee County, Wisconsin, and Cook County, Illinois, and at other points, chiefly in the middle west.

The total yardage involved in contracts for concrete pavements in 1912 was about 8,800,000 sq. yd., of which nearly 1,900,000 were for rural highways.

The total yardage of concrete pavements by the end of 1923 was over 420,000,000 sq. yd., of which nearly 300,000,000 were for rural highways. (Fig. 1.)

A somewhat better understanding of these qualities may be had if we reduce these sq. yd. figures to the equivalent length of pavement 18 ft. wide. (Table 1.) Thus in the short space of fifteen years, the concrete highways mileage has increased from three miles to over 25,500 miles, while the equivalent mileage for all construction, including alleys and streets, reduced to 18 ft. wide, makes at present a grand total of over 40,000 miles, with over 3,000 miles under contract at the close of 1923.

The various tables and diagrams in this paper give considerable statistical information regarding the extent and distribution of concrete highway construction throughout the United States.

Table 1.—Concrete Highway Mileage Completed During each Year for the Period 1919–1923 Inclusive

(Sq. yd. completed during each year reduced to equivalent mileage of pavement 18 feet wide)

State	Built 1919	Built 1920	Built 1921	Built 1922	Built 1923	Total to Jan. 1, 1924, including Work Prior to 1919
Alabama	....	....	4.2	7.1	5.5	40
Arizona	11.4	15.5	140.4	155.0	50.7	414
Arkansas	0.8	9.3	46.7	14.3	34.3	160
California	209.0	403.0	339.0	299.0	164.1	3,288
Colorado	3.8	27.8	29.2	37.1	27.6	137
Connecticut	14.7	25.2	46.7	34.4	42.3	249
Delaware	12.1	....	49.5	67.0	78.7	292
Dist. of Columbia	1.5	....	0.5	0.5	....	8
Florida	5.1	0.7	....	30.8	48.3	106
Georgia	51.2	43.8	27.7	37.8	54.0	310
Idaho	....	....	8.5	12.7	0.1	36
Illinois	190.5	340.0	431.6	625.0	1,041.0	2,991
Indiana	45.0	110.1	206.0	177.1	265.0	1,151
Iowa	2.6	43.4	158.5	100.1	95.0	439
Kansas	7.0	36.0	80.0	154.0	93.2	407
Kentucky	11.2	6.8	6.0	11.4	23.3	103
Louisiana	....	....	....	....	7.2	14
Maine	0.3	2.1	18.1	14.5	10.8	65
Maryland	54.7	42.0	94.5	84.9	129.2	863
Massachusetts	13.4	17.1	36.8	30.1	27.8	188
Michigan	98.0	83.5	325.0	234.0	319.0	1,466
Minnesota	20.4	68.6	100.0	104.8	77.5	446
Mississippi	6.4	19.4	42.7	29.4	46.6	172
Missouri	....	9.1	45.7	82.5	139.5	327
Montana	....	18.4	8.5	....	....	27
Nebraska	1.1	8.6	2.6	17.7	18.9	55
Nevada	6.8	....	4.8	2.9	2.4	29
New Hampshire	....	....	0.2	3.7	2.9	8
New Jersey	60.2	43.6	89.5	111.9	117.1	530
New Mexico	2.8	13.2	4.0	18.3	7.0	59
New York	99.6	176.5	507.0	329.0	397.0	2,243
North Carolina	14.2	5.0	43.0	127.0	336.0	595
North Dakota	....	....	3.2	0.7	0.5	4
Ohio	66.0	89.5	246.2	185.5	245.0	1,403
Oklahoma	12.0	14.5	70.7	75.0	42.6	238
Oregon	....	7.6	56.8	44.2	34.6	199
Pennsylvania	143.3	380.0	615.0	461.0	365.0	2,083
Rhode Island	6.2	....	4.0	5.2	7.0	33
South Carolina	13.9	11.5	23.9	41.0	20.5	140
South Dakota	....	....	....	....	0.9	1
Tennessee	4.2	2.2	14.2	....	16.8	58
Texas	50.6	26.6	110.0	90.7	55.0	366
Utah	....	60.0	18.9	42.2	10.7	217
Vermont	1.8	....	5.7	....	1.0	8
Virginia	65.2	57.7	51.5	37.9	114.0	497
Washington	75.0	201.0	149.0	118.5	91.5	980
West Virginia	45.7	50.5	65.0	56.6	94.5	517
Wisconsin	109.1	94.8	316.0	330.0	432.0	1,652
Wyoming	1.5	0.8	2.5	....	....	13
	1,538.3	2,565.4	4,649.5	4,442.5	5,193.6	25,627

AWARDS FOR CONCRETE ROADS.

There are some very interesting data as to the contracts awarded for concrete roads.

To accomplish the construction of 40,000 miles of concrete roads and streets, the greater portion of which has been built within the past five years, has, of necessity, developed contracting and engineering organizations to an extent little anticipated at the time when a satisfactory annual accomplishment of the Massachusetts State Highway Commission in the 90's was the completion of fifty miles of macadam road, the contracts for which averaged about one-half mile in length.

We contrast this with a letting by the Pennsylvania State Highway Department on April 18 and 19, 1922. On the first day thirty-two projects were awarded with a total length of 116 miles; and on the second day twenty-seven projects were let with a total length of 101 miles, which, with the exception of two or three miles, were for concrete roadways. The total amount of these contracts was approximately $9,148,000.

On Oct. 4, 1921, bids were received by the Illinois State Highway Commission for forty sections of pavement, aggregating 188 miles in length, including over a million yards of excavation, and 68 bridges using upwards of 15,000 cu. yd. of reinforced-concrete.

Some appreciation of the engineering organizations that is necessary to prepare for and handle a letting of this sort is gained from some of the details, which were kindly furnished by Clifford Older, Chief Highway Engineer for Illinois. For example, there were a total of 2664 printed proposals furnished to prospective bidders, or an equivalent of 1¼ acres of printed paper. There were 961 of these proposals returned as bids, the opening and reading of which consumed approximately eight hours. The blueprints of the plans for this letting used 12¼ miles of blueprint paper, twenty-four inches wide.

Two contracts for concrete highways awarded by Maricopa County, Arizona, stand out as being perhaps the largest single contracts for any high-class type of pavement. One of these contracts was for 127.4 miles, on which construction began November 1, 1920, and one for 167.5 miles, construction on which was begun March 20, 1922. Both of these contracts were let to the same con-

tractor and both contracts were finished on time. The first required almost fifteen months for completion and the second a little over twelve months.

Among other large contracts for concrete roads may be noted one for fifty miles in Phillips County, Arkansas. In 1922 and 1923, there were sixty contracts awarded for ten miles or more, six of which were for twenty-four miles or over. These contracts were scattered through the states of Illinois, Oklahoma, Iowa, Indiana and Kansas.

TABLE 2.—SHOWING AVERAGE SIZE OF CONTRACTS FOR CONCRETE PAVEMENT AWARDED DURING LAST FIVE YEARS

Year	Pavement	Number of Contracts	Total Yardage	Average Yardage
1919	Roads	1,271	41,335,342	32,522
	Streets	1,064	11,086,419	10,429
	Alleys	309	1,038,173	3,360
1920	Roads	846	29,326,689	34,665
	Streets	957	8,814,782	9,211
	Alleys	265	907,164	3,423
1921	Roads	1,486	43,862,503	29,517
	Streets	1,290	10,695,548	8,291
	Alleys	398	1,606,085	4,035
1922	Roads	1,866	58,301,413	31,244
	Streets	2,172	18,607,792	8,567
	Alleys	562	2,176,500	3,871
1923	Roads	1,825	50,893,999	27,887
	Streets	2,737	24,385,497	8,910
	Alleys	822	2,658,276	3,234

Table 2 shows the average size of contracts for concrete pavements awarded during the last five years. As would be expected, the awards for highways are much larger than those for street or alley improvements. It would seem that for these five years the average size of highway contracts was approximately 30,000 sq. yd., or about three miles; and it is to be noted that the average size of contract in any one of these five years does not vary markedly from the general average for the five-year period.

A more significant compilation of data respecting concrete road contracts is found in the diagram, Fig. 2, which is a graphical

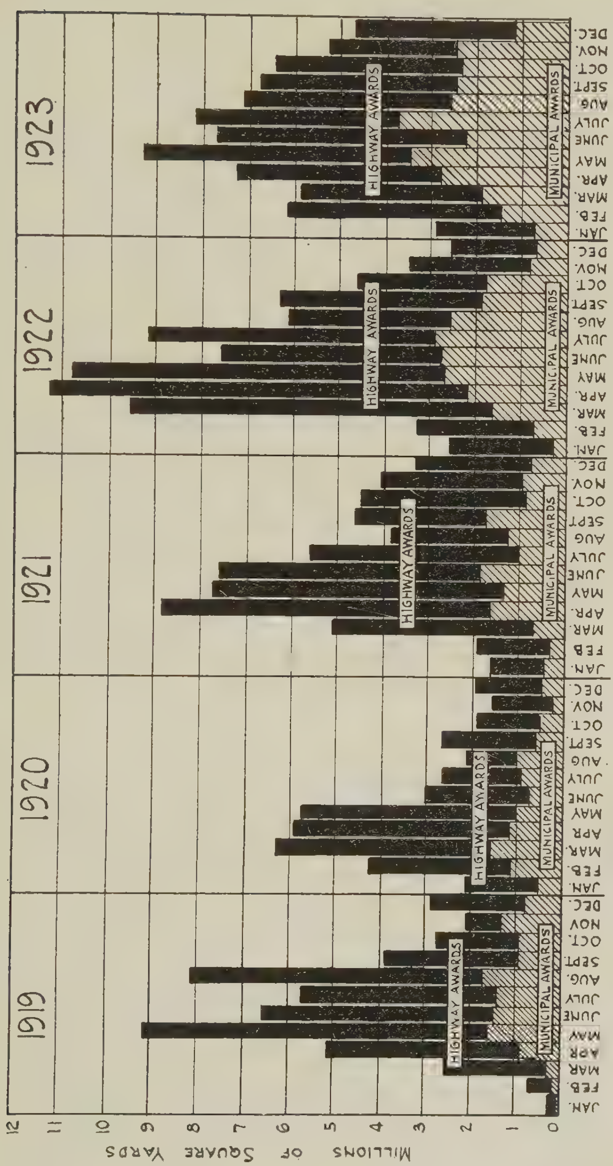


FIG. 2.—AWARDS OF CONCRETE PAVEMENT, 1919-1923.

presentation of yardage of concrete pavements awarded each month from January, 1919, to December, 1923. This diagram is worth a little study.

Owing to the weather conditions prevailing in a large part of the United States, highway construction is, of necessity, a seasonal occupation. Little or no work is done in the winter months, but with the great increase in the amount of work to be done, and the more extensive contracting organizations developed, it has been organized so as to make possible considerable work throughout the year, particularly in the preparation and storage of materials. This had led, on the part of the engineers, to the offering of work for bids in the late summer and early fall in order that the contractors may know with sufficient time ahead to plan their coming season's work several months earlier than had been, until quite recently, the custom in highway construction. From this viewpoint, (Fig. 2) is of special interest as indicating a greater proportional amount of work awarded in the fall months of 1921-22 and 1923, as compared with awards made in 1919 and 1920. It is clear that such an arrangement must make for more efficient and economical handling of the large organizations the contractors must necessarily maintain, with corresponding advantage secured by the public by the more expeditious and economical methods thus made possible. (See also Fig. 3.)

DEVELOPMENT OF CONCRETE ROAD-MAKING MACHINERY.

The development of road-making machinery in the past decade has been rapid, as might be expected from the extraordinary amount of road building that has been accomplished during this period. If dependence upon man power and animal power had been necessary to the same extent as was used but a few years ago in the road-building operations, unless it had been possible to devise machinery and mechanical appliances to do a major portion of the work, the large road building programs which have been successfully carried on in so many sections of the country would have been wholly impossible. But mechanical appliances have been devised, so that we may see today a concrete road under construction in which nearly every man employed is operating some piece of machinery, the actual man power having been reduced to a minimum.

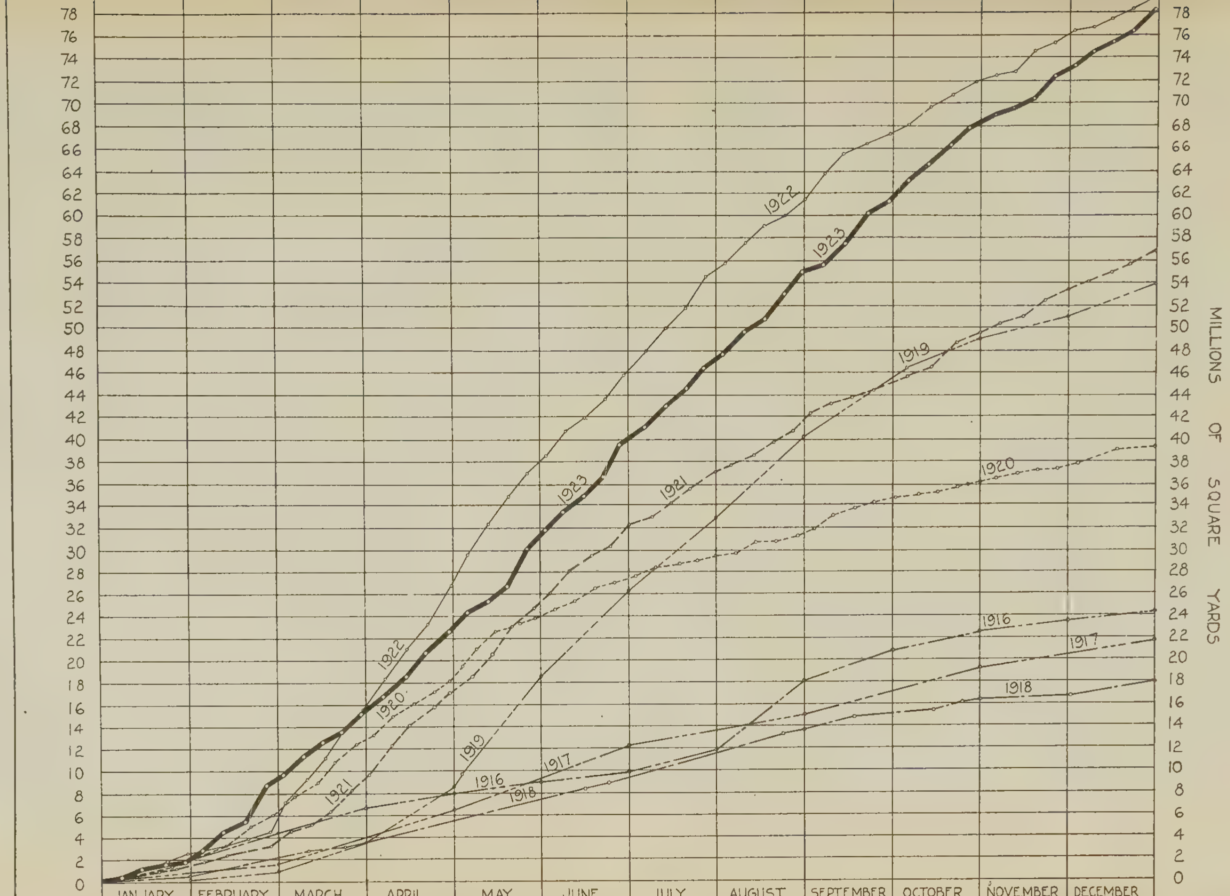


FIG. 3.—COMPARATIVE YARDAGE OF CONCRETE PAVEMENT AWARDED DURING THE YEARS 1916 TO 1923.

A good rate of progress for a two-sack batch paving mixer is about 1200 sq. yd. per eight-hour day, or about 600 ft. of 18-ft. width pavement, averaging 7 in. thick. A few records made on work that was especially well planned as to machinery required to handle material and distribute the concrete will be of interest.

In 1919 G. P. Scharl, a contractor in Branch County, Michigan, laid 1000 ft. of 16-ft. concrete road per eight-hour day. So far as can be learned, Mr. Scharl was the first to arrange his job so as to achieve this output.

The following year two contractors in Minnesota and two in Illinois made similar records, while Mr. Scharl beat his own. The records of a number of contracts for the year 1920 ran well over 1000 ft. per day.

In 1923 the largest day's output recorded was announced by the Illinois Division of Highways as 2017 ft. of 18-ft. wide pavement, 6 in. thick for 14 ft., spreading to 9 in. thick at the edge in the outer 2 ft. on both sides. This record was made with a 32-E Koehring mixer, fed by industrial train outfit.

To account at all exhaustively, or in any considerable detail, the development of the great variety of machinery and appliances that are now used in concrete road construction would call for much more time and space than is at the disposal of the writer, who is obliged to leave the discussion of this phase of his subject to someone else to develop.

Improvement in the finishing of concrete roads has been marked and has been brought about chiefly through the employment of finishing devices, some of which have been particularly effective, although very simple and cheaply constructed.

Canvas belts, with which everyone connected with highway construction is familiar today, were first used to finish concrete pavements in 1915 in California, and in Wayne County, Michigan.

A marked improvement in the finishing was the introduction of the light roller which removed the excess water, and, at the same time, ironed out any inequalities, thus producing a surface free of small uneven places that would be noticed by traffic, as was the case with the hand trowel method of finishing the earlier concrete roads.

Although rollers had been used to some extent for finishing concrete surfaces, they first attracted attention as a method of

finishing concrete pavements in Macon, Ga., where they were employed under the direction of City Engineer, J. J. Gaillard, about 1916.[13] This method rapidly gained the favorable acceptance of engineers all over the country, and it was not more than two, or at the latest, three years, when we find this provided for in most state highway and other highway specifications. However, because of certain patent rights and resulting infringement suits, other devices have been used successfully by many contractors and engineers, accomplishing much the same result as the roller.

Machines for finishing concrete roads have been developed and are used with success. In fact, the specifications of some State Highway Departments are so drawn that it would be rather expensive and awkward to finish a concrete highway except by a finishing machine. One of the earliest of these was built and operated by R. D. Baker, of Detroit, in 1913, and was used upon Pasadena Street, Highland Park, Michigan.[14] This machine was essentially a mechanical troweling machine, the steel plates or trowels moving on a chain traveling across the pavement and were guided by channel irons which were bent to conform to the crown of the road. These channel irons were mounted on four wheels riding on the curb. It was pushed by hand and the chain carrying the trowels also was operated by hand. Later improvements in this machine included a small gasoline motor, by which it was operated. A similar machine was also used by Mr. Baker on some of the Wayne County, Michigan, roads.

A finishing machine consisting of a concave roller was devised by Henry Bowlby, then State Highway Engineer of Oregon and was used in 1914 to finish some of the Oregon concrete pavements.[15] Neither of these machines came into general use.

Another machine quite different in principle was developed in 1913 by E. G. Carr,, of California. This machine strikes the concrete to proper shape, and, at the same time, tamps the surface. Many improvements have been made upon the original model from which has been developed what is now the well-known Lakewood finishing machine, probably more widely used than any other. This apparatus has a framework resting upon wheels which are guided by the side forms and upon which they rest. Suspended to the framework are two moving templets cut to the width and crown of the road. The forward templet has a horizontal movement of a

few inches back and forth across the road, while the other templet has a reciprocating vertical motion. The first strikes the concrete to proper shape, while the second tamps the surface to compact it. The machine moves ahead under its own power furnished by a gasoline engine mounted upon the frame. It is operated by one man.

A similar machine, without the tamping feature, but with a specially arranged striking templet, has been recently perfected by William Ord, and has been put on the market by A. W. French & Co., of Chicago.

SPECIFICATIONS.

The specifications for concrete roads have undergone many changes in order to keep them abreast of the improvements which experience has demonstrated to be productive of better results.

The specifications of twenty years ago for laying a concrete pavement were very crude as compared with the specifications for the modern concrete highway. In a treatise on highway construction, published in 1900 by Austin T. Byrne (p. 314) occurs the following specification for hydraulic cement pavement, which is to be laid on 14 in. of compacted cinders. This construction was used in several cities as paving for alleys. It was essentially a modification of sidewalk construction; a lean base covered with a thin top course of a richer mix.

"Upon a foundation so formed shall be laid a concrete composed of one part of portland cement (either *Hilton or Manheimer Brand*) and three parts of clean, sharp, coarse sand, thoroughly mixed dry and made into mortar with the least quantity of water, and thoroughly intermixed with broken stone or furnace slag in such quantity (about seven parts) that, when tamped or rammed solidly in place free mortar will rise to the surface and exhibit a depth of 3 in. of the said concrete. Upon this concrete foundation a surface mixture shall be laid 1 in. in thickness, composed of one part of portland cement (*Dykerhoff or Star Stettin Brand*) and two parts of crushed granite, with just sufficient water to make a stiff mortar; this surface coat shall be thoroughly compacted by tamping; and shall be dressed with a small quantity of dryer, composed of one-half pure cement and one-half flint sand floated over the entire surface as a finish."

From this time, a great variety of specifications for concrete highways was developed by various engineers. There was little to guide them. Most engineers then required a two-course type of construction, the base course 5 or 6 in. thick and of very lean mix, the wearing surface 1 or 2 in. thick of a richer mixture.

The earlier pavements laid in Bellefontaine, Ohio, and Richmond, Indiana, which have been previously mentioned, were built under such specifications.

No attempt toward formulating a standard specification for concrete roads had been made up to 1909, when, at a meeting of this society, a committee appointed for this purpose made a report and presented such a specification. This provided for a 6-in. base of 1 : 3 : 5 concrete, with a 4-in. wearing surface of 1 : 3 : 6 to be placed upon the base within fifty minutes after the concrete for the base had been mixed. "Single coat" work, i. e., one course construction, is mentioned, with the provision that it shall be proportioned 1 : 2 : 3, tamped and struck off, then "the coarser particles of the concrete shall be tamped to a depth which will permit a mortar finish." [16]

The specification for the fine aggregate was not greatly different from that of the present-day practice. It should be evenly graded material, from the ¼-in. size to that retained on 100-mesh sieve, not more than 6 per cent to pass the latter. Coarse aggregate was to be over ¼-in. size and all retained on a ¾-in. ring. It will be noted that the upper limit for the coarse aggregate is very much smaller than is the practice today, and that the mix for the top course is much leaner.

Machine mixing is preferred "since a more thorough and uniform consistency can thus be obtained." Wood forms are specified, but no mention is made of reinforcement. The concrete was to be laid in rectangular slabs; cross forms are specified to be placed at right angles to the side forms.

Many of the changes that have taken place in specifications for concrete highways were the outgrowth of improved equipment. Thus the specifications for side forms call for increased weight and stability in order that they may be able to support the finishing machines, which require much heavier track for their guidance than the average plank side form, as was at first laid, would furnish. Likewise, the development of means to handle the aggregates made

it unnecessary to place them on the road bed, which has always been recognized as bad practice, but necessary in the earlier work. Now it is no longer necessary, and many state specifications specifically prohibit so placing the aggregates.

With the exception of a few of the earliest pieces of concrete placed for pavements, nearly all have been laid with machine mixing, rather than by hand mixing. Owing to the bad results that had been obtained from continuous mixers for concrete work for other purposes, practically from the beginning this type of mixer was not permitted under specifications for concrete highways, but in almost every instance it was required that the concrete be mixed with a batch mixer.

Investigations showed, particularly those carried on by Prof. Duff A. Abrams of the Lewis Institute, Chicago, that the time of mixing had an important influence on the strength of the concrete, and specifications were developed requiring a definite time, varying from three-quarters of a minute to two minutes, and often requiring that the mixer should have an automatic device to insure that the mixing would continue for the time prescribed.

The researches of Professor Abrams have exerted great influence upon the development of specifications for concrete highways.[17] Perhaps, the most striking of his results were those which came from the extensive series of tests which developed the fact that the quantity of water had a controlling influence upon the strength of concrete and that aggregates of different gradings affect the strength of the concrete, because of the fact that some gradings require more water to be used than others, to give a concrete of the requisite consistency for convenient handling upon the work.

What Professor Abrams calls the fineness modulus has resulted in making possible a rational use of a great variety of aggregates as to grading, which has been productive of great economies; and specifications of highway engineers have been modified accordingly.

In 1919 a summary of state highway specifications for concrete roads was made by the author and is here reproduced. This gives a general idea of the principal features that were included in the specifications and the variations that then existed. It will be noted that fine aggregate was specified by definite limits as to size in a large proportion of the specifications. A majority called

SHEET 1

STATE	DATE	FINE AGGREGATE			COARSE AGGREGATE			MIXING							CONSISTENCY		FORMS			REINFORCEMENT		
		Grading Specified by Sizes	Grading General Only	All to Pass Inches	French Coeff.	Max. Size Inches	Min. Size Inches	Cement Factor Bbl. per Cu. Yd.	Proportions	Time of Mix. Minutes	Materials to be Measured	Water to be Measured	Mixer Timing Device	Mixer Water Measuring Device	General Provision Only	Specific Requirements	Wood	Steel	To be Oiled or Soaped	Specified	Mesh Lbs. per 100 Sq. Ft.	Circumferencial Specified
Alabama	1918	—		1/3	8	2	1/3	base top	1-2-3	1	—	—	—	—	—		—	—				
Arizona	1919	—		1/4	10	2	1/4		1-2-4	1	—	—	—	—	—		—			When used	25	
Arkansas	1919	—		1/4	10	1¾	1/3		1-2-3	1	—	—		—	—		—	—	—			
California	1919	—		1/4		2½	1/4		1-2-4		—	—			—		—			See plans		
Colorado	1918	—		1/4		2	1/4	170	1-2-3	1	General	General			—		—	—				
Connecticut	1919	—		1/4		1½	1/4		1-2-4	1½	—	—	—	—	—		—	—	—	When used	25	
Delaware	1918	—		1/4	12	2	1/4		1-2-4	1½					—			—		See plans		
Florida	1919	—		1/4	10	2	1/4		1-2-3	1	—	—		—	—		—	—	—			
Georgia	1919	—		1/4	10	2	1/4		1-2-3	1	—	—	—	—	—		—	—	—			
Idaho		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
Illinois	1919	—	o	95%-1/4	6	G-2, S-2½	1/4	161	1-2-3½	1	—	—				—	—	—	—			
Indiana	1919	—		1/4	8	2	1/4	191	1-1½-3	1	—	—	—	—		—	—	—	—	When used	40	
Iowa	1918	—		1/4		1½	1/4	170	1-2-3	1½	—	—			—		—	—		Over 20 wide		
Kansas	1917		—	1/4	6 8	1½	1/4	140-161	1-2-3½	1	—				—		—	—		When used	45	
Kentucky	1918	—		1/4	10	2	1/4		1-2-3	1½	—	—	—	—	—		—	—		See plans		
Louisiana		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
Maine	1918	—		1/4		1½	1/4		1-2-4	¾					—		—	—				
Maryland	1918	—		1/4	10	2	1/4		G 1-1½-3, S-1-2-4	1	—	—	—	—	—		—	—	—	When used	22-25	
Massachusetts	1919	—		1/4		1½	1/4		1-2-4						—		—					
Michigan	1918	—		1/4		G-2, S-2¼	1/4		G 1-1½-3, S-1-1¾-3	1	—				—		—	—				
Minnesota	1919	—		1/4	8	2	1/4		1-2-3½ conc.	1	—	—	—			—	—	—		—		—
Mississippi		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
Missouri		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
Montana		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
Nebraska	1918	—		1/4	8	2	1/4	1.70	1-2-3	1	—	—			—		—	—	—			
Nevada	1919	—		1/4	8	2½	1/4		1-2-4	1	—	—		—	—		—	—				
New Hampshire	1918	—		1/4		2	1/4		1-2-3	1½	—	—		—	—		—	—	—	When used	25	
New Jersey	1918	—		1/4		1½	G-1/4, S-3/4	1.51-1.91	1-2-3	1½	—	—			—			—	—	When used	34	
New Mexico		—		3/8	8	2	1/4		1-2-3	1½	—	—		—	—		—	—	—	When used	25	
New York	1918	—		1/4		2¾	1/4		1-1½-3	1	General				—		General	Not	Specific			
North Carolina	1918	—		1/4		2	1/4		1-1½-3	1½			—	—	—		—	—	—	When used	25	
North Dakota		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
Ohio	1919	—		1/4	8	2½	1/4	1.75-2.0	1-1½-3	1	—	—	—	—	—		—	—	—	When used	28	
Oklahoma	1918	—	—	1/4	7½	2	1/4		1-2-3	1½			—	—	—		—	—		When used	25	
Oregon	1918	—		1/4		G-1¼, S-2¼	1/4		1-1½-3	1	—				—		General	Not	Specific	See plans		
Pennsylvania	1919	—		1/4	10	2	1/4		1-2-3	1¼	—	—	—	—	—		—	—	—	When used	25	
Rhode Island	1918										—				—		—					
South Carolina	1918	—		1/4		1¾	1/4		1-2-4	1					—		—	—	—	See plans		
South Dakota	1917	—		1/4		2	1/4	1.70	1-1½-3	1	—				—							
Tennessee		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
Texas	1919	—		1/4	8	2	1/4		1-2-3	1½	—	—	—	—	—		—	—	—	When used	25	
Utah	1918		—	1/4		2	1/4	1.60	1-1¾-3½	1	—	—			—		—	—		—		—
Vermont		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
Virginia	1918	—		1/4		2	3/4		1-2-4	1	—	—	—		—		—	—	—			
Washington	1918	—		1/4		3	1/4	1.66	1-2-3	1	—	—			—		—			When used	28	
West Virginia	1918	—		1/4		2½	1/4		1-2-3	1	—	—	—	—	—		—	—	—		25	
Wisconsin	1918	—		1/4	7	2	1/4	1.68	1-2-3½	1	—	—		—	—		—	—				
Wyoming	1919	—		1/4		2	1/4	1.70	1-2-3	1	—				—		—	—				

NOTE: States from which specifications were not available shown thus, o o o o. Otherwise, a blank space indicates that the specifications are silent on the particular subject indicated at top of column

FIG. 4.—SUMMARY OF STATE HIGHWAY DEPARTMENT SPECIFICATIONS FOR CONCRETE ROADS.

SUMMARY OF STATE HIGHWAY DEPARTMENT SPECIFICATIONS FOR CONCRETE ROADS
SHEET 2

State	Date	Expansion Joints					No Joints, End of Day Only.	Finishing							Trueness of Surface		Cold Weather		Curing				
		Specified	Spacing in Feet	Steel Protection Plates	Thickness of Material Inches	Details on Plans		Struck with Template	Split Template at Joints	Wood Floats	Surface Broomed	Roller and Belt	Split Float at Joints	Machine Finish	General Provision Only	Allowance of Variation Inch to Feet	Work Prohibited	Work Permitted with Precaution	Canvas Cover	Wet Earth	Ponding	Kept Covered Min Days	Closed to Traffic Min Days
ALABAMA	1918	—			1/8 - 1/2	—		—				—	—					—	—	—		18	21
ARIZONA	1919	—	30		1/8 - 1/2	—		—				—	—				—		—	—	—	10 - 18	13 - 21
ARKANSAS	1919	—	37½		1/8 - 1/2			—				—	—				—		—	—		7	14
CALIFORNIA	1919											—				1/4 5				—	—	10	
COLORADO	1918	—	30		1/8			—				—	—			1/4 -	—		—	—		10	14
CONNECTICUT	1919	—	25					—		—		—	—				—		—	—		8	11
DELAWARE	1918						—	—				—					—		—	—		14	30
FLORIDA	1919	—			1/8 - 1/2	—		—				—	—					—	—	—		18	21
GEORGIA	1919	—	33		1/8 - 3/8			—				—								—	—	18	21
IDAHO		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
ILLINOIS	1919						—	—				—		—		1/4 —	—		—	—	—	14	30
INDIANA	1919						—	—				—		—		1/4 —	—		—	—	—	15	21
IOWA	1918	—	36		3/8			—		—				—		1/4 - 6	—			—		10	14
KANSAS	1917	—	33⅓		1/4 - 3/8			—				—	—			1/4 —	—		—	—	—	14	30
KENTUCKY	1918	—			1/4 - 1/2	—		—				—	—			1/8 — 10; 1/4 — 3		—	—	—		14	17
LOUISIANA		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
MAINE	1918	—	30		1/2			—		—						1/4 —	—			—			
MARYLAND	1918						—	—	—			—					—		—	—		18	21
MASSACHUSETTS	1919	—						—		—	—				—			—	—	—		14	14
MICHIGAN	1918	—			1/4	—		—				—	—			1/8 — 15; 3/4 — 3			—	—		14	14
MINNESOTA	1919					—	—	—				—		—		1/8 — 10; 1/4 — 3	—		—	—	—	14	21
MISSISSIPPI		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
MISSOURI		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
MONTANA		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
NEBRASKA	1918	—	35		1/2							—	—				—			—		10	14
NEVADA	1919	—	25		1/4			—				—					—				—	12	21
NEW HAMPSHIRE	1918	—	40		1/4 - 1/2			—				—	—				—		—	—		18	21
NEW JERSEY	1918	—	36		1/8 - 3/8			—				—	—				—			—		10	14
NEW MEXICO		—	30		1/8 - 1/2			—				—	—				—		—	—		18	21
NEW YORK	1918	—	30		1/4 - 3/8			—		—	—		—			1/2 - 10		—		—		14	14
NORTH CAROLINA	1918	—	40		1/4 - 1/2			—				—	—				—		—	—		16	21
NORTH DAKOTA		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
OHIO	1919	—	40		1/4 1/2			—				—	—			1/2 — 10		—	—	—		10	21
OKLAHOMA	1918	—	40		1/4 - 1/2			—				—	—				—		—	—		18	18
OREGON	1918	—	30			—		—		—	—						—			—		10	21
PENNSYLVANIA	1919	—	200 to 500		1/2	—		—				—	—	—			—		—	—		18	21
RHODE ISLAND	1918	—	30		1/4 - 1/2			—		—	—								—	—		14	14
SOUTH CAROLINA	1918	—			1/4 - 3/8	—		—	—	—						1/4 - 3		—	—	—		10	14
SOUTH DAKOTA	1917	—	36		1/4								—			1/4 —		—		—		10	14
TENNESSEE		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
TEXAS	1919	—	40		1/4 - 1/2	—		—				—	—				—		—	—		18	21
UTAH	1918	—	40		1/4 - 3/8			—				—	—			1/4 —	—		—	—	—	14	30
VERMONT		o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o
VIRGINIA	1918	—	40		1/4 - 1/2			—				—	—				—		—	—		13	21
WASHINGTON	1918	—	30		3/8			—				—	—				—			—		10	30
WEST VIRGINIA	1918	—	30		1/4 - 1/2			—				—	—					—		—		10	14
WISCONSIN	1918	—			1/4 - 3/8			—	—	—		—	—			1/4 —		—	—	—	—	11	14
WYOMING	1919	—	30		1/4			—				—	—			1/4 —	—		—	—		10	14

NOTE: States from which specifications were not available shown thus, o o o o
Otherwise, a blank space indicates that the specifications are silent on the particular subject indicated at top of column

FIG. 4.—(*Continued*).

for 1 : 2 : 3 mixture; all provided for machine mix. The time of mixing was prescribed in a large number of instances at one minute, while ten call for a minute and a half, and only one as short a time as three-quarters of a minute. In but three states was there a specific requirement as to consistency. Most required that materials should be definitely measured. The construction of expansion joints at intervals, varying from twenty-five to forty feet, was required in most instances; but four states specified that no expansion joints were to be constructed. Specifications for finish, in general, were framed to insure a dense and true surface and the majority had adopted the roller and belt method, while five provided for machine-finished surface. Either no provisions were made, or they were rather indefinite as to the requirements for variations that would be permitted for a true surface. Curing was generally done by covering with wet earth; in some instances ponding was prescribed. The length of time that the concrete should be cured varied from eight days to two weeks, and traffic was prohibited from eleven to thirty days.

This summary of the state highway specifications prepared in 1919 is of particular interest when compared with a similar study of highway specifications made in 1923 by C. R. Ege of the Portland Cement Association. We now find that the time of mixing is, in no instance, less than one minute; eleven specify one and one-half minutes; two specify one and one-quarter minutes; the others providing one minute. But two states specify no expansion joints, while twenty-four call for them, at intervals varying from 30 to 100 ft. and from ¼ to 1 in. in thickness. Practically all the specifications permit the use of the roller for finishing the surface; while those providing for machine finish total twenty-nine. Definite provisions as to the trueness of surface now appear in the 1923 specifications. No marked changes appear in the specifications for curing.

The colorimetric test which was not mentioned in specifications four years ago is now required in eleven state specifications. This test, which was devised at Professor Abrams' laboratory, is a very simple method of detecting organic impurities, particularly in fine aggregate. It is one of the most important safeguards to be taken, for it has been definitely demonstrated that only a very slight amount of organic impurity, which would not be detected

except by some chemical test, as the colorimetric test, is sufficient utterly to destroy the strength of the concrete. This test can be readily made in the field, requiring merely a 3 per cent solution of sodium hydroxide in which the fine aggregate is shaken and allowed to settle. If the solution clears, without sensible color remaining, it indicates freedom from organic impurities. Should the liquid turn a reddish brown or deep amber color, it indicates an amount of organic impurity that would be detrimental, and the aggregate should not be used unless it be washed to free it from such impurity.

Also it is to be noted that far greater attention is being given to the reinforcement of concrete highways. While but two states required reinforcement in 1919, nine do at present.

INVESTIGATIONS AND RESEARCHES.

The development of concrete roads has been attended and greatly accelerated by scientific research. It is not too much to say that the investigations growing immediately out of concrete road construction have inaugurated a program of research of greater breadth than has been the case with any other type of paving construction up to the present time.

The researches have been carried on by a large number of widely dispersed agencies, the more important of these including the U. S. Bureau of Public Roads, various state highway commissions, and many universities.

Highway research work, while of comparatively recent date, has already reached a position of prime importance in the engineering research program. This has been possible through organized effort brought about through the active interest of such associations as that of the American State Highway Officials, and the Advisory Board on Highway Research of the National Research Council—all working in closest co-operation with the U. S. Bureau of Public Roads. This co-operation has resulted in increased inspiration to the individual agencies, and there has been avoided any stifling of initiative that is almost inevitable to result from any centralized control of such activities. Thus it is that all the conditions being so favorable and the time opportune, we find so great an amount of valuable research work has been done in connection with the construction of concrete roads.

As with other phases of the development of concrete roads, the limits of this paper will not permit any attempt to present or even mention all of the various researches that have been undertaken. These have developed from somewhat crude and unscientific experimental sections of roads, to the building of comparatively long stretches of roads with highly specialized apparatus for most thorough scientific and mathematical investigations of the action of the road slab under traffic, supplemented by exhaustive laboratory studies.

Among the earlier experiments were those with oil concrete, described in the *Transactions,* Am. Soc. C. E. Vol. LXXIV, December, 1911, p. 273. Theodore S. Oxholm arranged with the late L. W. Page of the Bureau of Public Roads to build a section of this type of pavement in the Borough of Richmond on Innis St. between John St. and Morning Star Road.

The concrete was mixed in the proportion of one part cement, two parts sand, and four parts broken stone (¾ in.) with the addition of oil to the amount of 15 per cent of the weight of the cement.

"The cement and sand was mixed and water added to form a mortar, after which the oil and then the stone was added. The whole mass was then thoroughly mixed and deposited on the street to form a wearing surface 4 in. in thickness.

"I have made a recent examination of the pavement and found that the joint at the end of each day's work is very marked."

A report on this pavement dated Sept. 13, 1911, states ". . . a much better piece of work than this could be done with the knowledge gained, both as to the materials to be used and the methods of laying and finishing each day's work. I do not think, however, that this class of pavement, after fifteen months in use, has shown sufficient merit to make it advisable for us to lay any more of it at the present time."

Among the more notable of the earlier experimental roads should be mentioned one built by the U. S. Bureau of Public Roads at Chevy Chase.[18]

The laboratory investigations of concrete, as a material of construction, have been under way for a considerable period, but have been greatly accelerated in the past two decades. This sub-

ject is more particularly treated in Dr. Talbot's paper presented at this convention.

The greater amount of research work that has had immediate application to the use of concrete in highway work is under way in the laboratories of the U. S. Bureau of Public Roads and the Lewis Institute at Chicago; while the notable work done at the University of Illinois and the University of Wisconsin investigating the uses of concrete in general construction has been of great influence and assistance to those at work on concrete road studies.

The investigations of concrete roads have brought about the special development of drills suitable for cutting samples from the roads themselves, so that there is made possible laboratory investigations upon the concrete that has been actually in use in the highway under various conditions and for different lengths of time. Already fourteen state highway departments, two counties, in addition to the U. S. Bureau of Public Roads and a number of cities, and some universities, are equipped with these outfits; and thousands of cores have been taken from concrete highways in many sections of the country. The data from these investigations, as they become available, must prove of great value.

Among the more elaborate field investigations, a few stand out as notable for the extent and thoroughness of the studies made. These are the Pittsburg Test Road at Pittsburg, California, built in 1921 and 1922; the Bates Experimental Road at Bates, Illinois, constructed in 1921 to 1923; and the circular track and impact tests of the U. S. Bureau of Public Roads, which have been carried on since 1920; the circular track being built in 1922.

Pittsburg Road Test (Pittsburg, Calif.).—A complete report of this remarkable highway research was published in 1923 by the State Highway Commission of California, of which A. B. Fletcher was chief engineer. The beginning of this work was undertaken by the Columbia Steel Company of Pittsburg, Calif., in 1921, the U. S. Bureau of Public Roads and the State Highway Commission of California having informal observers present to report on the progress made. It was the original plan of the Steel Company to investigate the effect from the use of steel reinforcements in concrete roads. But it soon became apparent that the possibilities were far greater than had been anticipated. To realize them, however, would require expenditures that the Steel Company was not

CHARTED SUMMARY OF STATE HIGHWAY DEPARTMENT SPECIFICATIONS FOR C

STATE	DATE REVISED	FINE AGGREGATE			COARSE AGGREGATE					AGGREGATE ON SUBGRADE	MIXING						CONSISTENCY		FORMS			JOINTS			
		NUMBER OF SCREEN SIZES SPECIFIED	COLORIMETRIC TEST SPECIFIED	MAXIMUM SIZE (INCHES)	FRENCH COEFF	MAXIMUM SIZE (INCHES)	MINIMUM SIZE (INCHES)	USE SLAG	USE PIT RUN		CEMENT BBLS. PER CU. YD.	PROPORTIONS	MIXING TIME MINUTES	TIMING DEVICE REQUIRED	MATERIALS TO BE MEASURED	WATER TO BE MEASURED	GENERAL PROVISIONS ONLY	SLUMP TEST REQUIRED	WOOD	STEEL LBS PER FT	OILED OR SOAPED	EXPANSION SPECIFIED	WIDTH OF EXPANSION (INCH)	SPACING (FEET)	LONGITUDINAL CENTER JOINT
ALABAMA		4		¼	8	2	¼	—				1:2:3	1	—	—	—		—	—	—	—	—	¼-½	0	
ARIZONA		4		¼	8	2	¼					1:2:3½	1		—	—	—		—	—	—	b			
ARKANSAS		4		¼	7	2½	¼		X	X		1:2:3	1½		—	—		—	c	—	—	—	½-1	60	for 18 ... wider
CALIFORNIA		3		¼	10	2½	¼			c	1.5	1:2:4	1	—	—	—	—		—	—					—
COLORADO		4		¼	10	2	¼		X		1.7	1:2:3	1½		—	—	—		—	—		—	¼	30	
CONNECTICUT		3		¼	8	2½	¼	CRUSHED ROCK ONLY		X		1:2:3	1½		—	—	—		—	7	—	—	½	40	
DELAWARE		3		¼	10	2¾	⅝	—		X		1:2:4	1½	—	—	—	—		X	8	—	a			—
FLORIDA		5	—	¼	8	2½	¼			X		1:2:4	1	—	—	—	—		—	—	—	a	¼-½	40	
GEORGIA		2	—	¼	10	2¾	¼	—		X		1:2:4	1		—	—		—		—	—	a	1		b
IDAHO		4		¼	10	3	¼			—	1.66	1:2:3	1	—	—	—	—		—	—	—	b	½	a	
ILLINOIS		4		¼	8	2½	¼		X	X	1.61	1:2:3½	1	—	—			—		—	—				—
INDIANA		4		¼	7	2½	¼		X	c	1.70	1:2:3	1	—	—	—		—		9	—				
IOWA		2	—	.185	8	2½	.185		—	X	SEE NOTE A		1		— W	—		—	—	—	—	a			
KANSAS		3	—	¼	6	2	¼		X	X	1.61	1:2:3½	1	—	—	—		—		—	—	a	½	35	—
KENTUCKY		4		¼	3.3 / 6.6	2½	¼		X	—	1.5	1:2:3½	1¼		—	—		—	—	—		—	½	35	—
LOUISIANA																									
MAINE		3		¼	6.67	2	¼		X	X	1.61	1:2:3½	1	—	—	—	—		X	—		—	¼	40	—
MARYLAND		3		¼	10	2¾	⅜					1:2:4	1	—	—	—		—	X	—	—				
MASSACHUSETTS		4		¼	8	2	¼				1.50	1:2:4	1½	—	—	—	—		—	—		—	½	60	—
MICHIGAN		5		¼	7	2½	⅛		X		1.54	1:2:3½	1	—	—	—		—		8½	—	a	⅜-¾	••	—
MINNESOTA		4	—	¼	8	2½	¼		X	X	1.54 / 1.44	•1:2:3½	1	—	—	—		—	—	—	—	b			
MISSISSIPPI		3		¼	8	3	¼	c	X	X		1:2:3 a	1	—	—	—		—	—	—	—	—	¼-½	a	
MISSOURI		4		¼	7	2½	¼		X	c	1.61	1:2:3½	1	—	—	—	—	—	c	—		b	½ per 100	••	—
MONTANA		4		¼		3	¼		X		1.7	1:2:3	1		—	—	—		—	—	—	—	⅜	30	—
NEBRASKA		3	—	¼	10	2	¼		—		1.74	1:2:3	1		—	—		—	—	—	—	X			a
NEVADA		3		¼	8	2½	¼					1:2:4	1		—	—		—	—	—		—	1	100	
NEW HAMPSHIRE		3		¼	8	2	¼		X			1:2:3	1½	—	—	—	—		—	—	—	—	⅜-½	40-60	
NEW JERSEY		6		¼	4% of wear	3	½			X	1.51 / 1.68	1:1½:3 / 1:1¾:3½	1¼	—	—	—		—		Max 9½	b	b	½-¾		a
NEW MEXICO		3		¼-⅜	8	2½	¼		X	X		1:2:3	1½		—	—	—		X	—	—	b	¼-½	30	
NEW YORK		3		¼	7°	2¾	¼	—	X	X		1:1½:3½ / 1:2:4	1	—	—	—	—		ON CURVES ONLY	—	—		½	40	—
NORTH CAROLINA		3		¼	10	2¼	¼			X		1:1½:3 / 1:2:4	1½	—	—	—	—		On curves 150' R	—	—				
NORTH DAKOTA		3		¼	8	1½	¼					1:2:3	1½	—	—	—	—		—	—	—	—	¼-½	a	a
OHIO		5		¼	7-8	1½	¼	—	X		1.75 to 2.0	1:1½:3	1	—	—	—		—		—	—	—	Designated by Engineer		
OKLAHOMA		3		¼	7	1½	¼		X		1.74	1:2:3	1½		—	—	—			—	—	—	Designated by Engineer		for Bates ty...
OREGON		2		¼	8	3	¼				1.66	1:2:3	1	b	—	—		—	—	—		—	⅜	30	b
PENNSYLVANIA		3		⅜	8	2¾	⅝	CRUSHED ROCK ONLY		X		1:2:3	1¼	—	—	—	—		ON CURVES ONLY	6½	—	a	½	a	b
RHODE ISLAND		4		¼	8	2	¼					1:2:3	1	—	—	—	—			—	—	—	½	a	—
SOUTH CAROLINA		3		¼	8	3	¼					1:2:4	1		—	—	—		—	—	—	—	¼-½	40	
SOUTH DAKOTA		4	—	¼	8	2½	¼			—	1.54	1:2:3½	1	—	—	—		—	—	—	—				—
TENNESSEE		4	—	¼	8	2	¼					1:2:4	1	—	—	—		—	—	—	—	—	¼-½	a	a
TEXAS		3		¼	8	2	¼		X			1:2:3½	1	—	—	—		—	—	—	—	a	¼-½	a	—
UTAH		4	—	¼	8	2	¼	—	X	X	1.7	1:2:3	1		—	—		—	—	—	—	—	⅜	40	—
VERMONT		4		¼	7	3	¼				1.5	1:2:4	1		—	—		—		—	—	—	½	30-50	a
VIRGINIA		4		¼	8	2½	¼					1:2:4	1	—	—	—		—	X	8	—	—	¼-½	a	
WASHINGTON		3	—	¼		3	¼			—	1.66	1:2:3	1	—	—	—	—		—			—	¼-⅜	20-30	a
WEST VIRGINIA		4		¼	4% of wear	2½	¼	—	X	—		1:1½:3	1	—	—	—		—	—	—	—	b	¼-⅜	a	a
WISCONSIN		4	—	¼	5	2	¼				1.5	1:2:4	1	†—	—	—	—		—	—		†—	¼-⅜	50	—
WYOMING		3	—	¼		2	¼	X	X		1.7	1:2:3	1		—				—			—	¼	30	

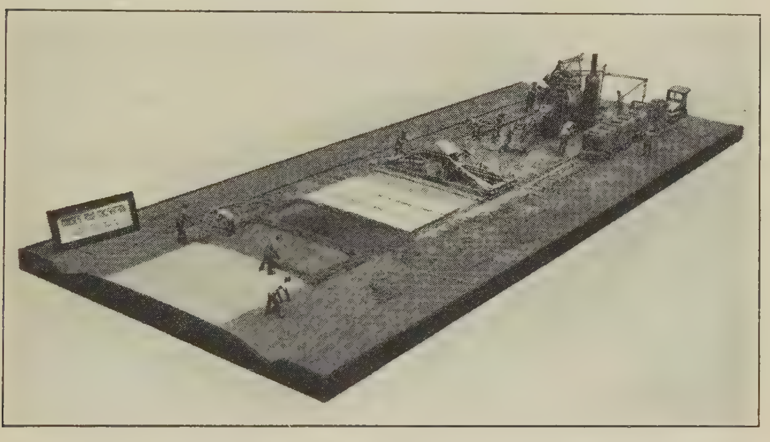
Notes:- a - As shown on plans.
b - Where required.
c - With special permission

— Required or allowed.
X Specifically prohibited.
Blank space Not mentioned in spec.
W Measurements by weight

• 1:2:4 W
•• Expansio
and ends

Note A: (Iowa) Proportions by weight table furnished in specifications.
⊙ (New York) For Fed. Aid Contracts

in a position to make. Thus, it came about that early in 1922 a formal co-operative financial agreement was made between the State Highway Commission of California and the U. S. Bureau of Public Roads to complete the tests and publish the results. A concrete track was built which was oblong, with two straight stretches of 450 ft. each, with semi-circular turns at each end of 75 ft. radius. There were thirteen sections or varieties of construction. Traffic consisted of loaded trucks, which were run around the track in opposite directions, developing in eighty-nine days a total traffic of 7.36 millions of tons.



CONCRETE ROAD MODEL OF THE U. S. BUREAU OF PUBLIC ROADS.

A particular feature of the test was the construction of observation tunnels in which to measure the flexure of the concrete pavement under various conditions of loading.

The Bates Road Test at Bates, Illinois.—This road is located about fifteen miles from Springfield on the Wabash Railroad and is the most extensive single experimental road that has been undertaken.

A stretch of road two miles in length was constructed, and not only many varieties of concrete construction were used, but also many other types of paving.

The traffic consisted of loaded trucks, the weight gradually increasing as the test progressed. The test itself was originated by the Highway Division of the Department of Public Works of the State of Illinois, under the immediate direction of Mr. Clifford

Older, Chief State Highway Engineer; the U. S. Bureau of Public Roads collaborating in this work.

The entire expenditure for this project has been estimated to be approximately $235,000. In a letter dated Jan. 9, 1924, Mr. Older states that on contracts awarded by the State at the present time, the saving that has been made, due to changes in the design of the roads as a result of the Bates Road Test, has amounted to nearly $5,000,000; and unless future developments show this design is faulty, the future paving program of Illinois, which is already assured, will make an additional saving of about $3,500,000. Perhaps, no more dramatic climax to a highway research is to be recorded.

A full description of the Bates Road Test will be found in the February, 1924, *Proceedings* of the American Society of Civil Engineers, which will contain a paper on this subject by Mr. Older.

Arlington Road Tests.—At the experimental farm of the Department of Agriculture, Arlington, near Washington, the U. S. Bureau of Public Roads has carried on for the past ten years an extensive series of tests, which has included most elaborate studies of concrete as a road pavement material.

A large number of slabs, 7 ft. square, have been constructed and tested under a great variety of specially controlled subgrade conditions to determine the effect of impact. A prominent feature in connection with these investigations is a circular track 200 ft. in diameter, composed of some sixty-three different concrete sections, which have been under observations, particularly as to the effect of wear. A full description of these tests may be found in the 1919, 1920 and 1921 issues of *Public Roads* printed by the U. S. Bureau of Public Roads; the *Proceedings* of the American Road Builders Association; and, more recently, a paper has been presented before the American Society of Civil Engineers (February, 1924, *Proceedings*).

All of these investigations are remarkable for the great care exercised in making the observations and the thoroughness with which they are reported. They have necessitated the design of some very complicated and extremely delicate apparatus, and they constitute a research of a very high degree of scientific attainment. This very brief mention of them gives a wholly inadequate idea of their extent and significance which can be obtained only by a close

study of the reports that have been published, and to which reference has been made.

The above tests have been primarily concerned with the proper design of concrete highways. There have also been undertaken a very important series of investigations which are concerned with the operation of highways, with the relative energy consumption that a sound basis for an economic theory for the development of highway systems may be evolved.

This class of investigations includes the measurement of resistance or comparative energy required under diffeernt conditions, and for different types of pavement construction. Very elaborate tractive resistance tests have been made. Tractive resistance has long been a study in connection with road work, and, among the earlier investigations of this character, it is very interesting to read the account of road resistance tests made by Sir John McNeill nearly a hundred years ago, who invented a specially designed wagon for making these tests, a full description of which will be found in Sir Henry Parnell's treatise on roads, p. 73, and in the appendix on p. 461.

Among the more recent researches of this character is a test conducted in 1917 by Prof. J. B. Davidson of the University of California at the university farm near Davis, measuring the pull in pounds per ton required to move a loaded wagon over eleven different varieties of road surfaces.[19]

A series of tests was made by the White Motor Co. in 1918 and described in *Engineering-News Record,* November 7, 1918, p. 843.

Bulletins 64 and 67 of the Iowa State College by T. R. Agg, published in 1922 and 1924, respectively, treat of the resistance to the translation of motor vehicles. This subject is one of special inquiry by one of the research committees of the Advisory Board on Highway Research of the National Research Council, of which Professor Agg is chairman.

The co-operative work at the Massachusetts Institute of Technology during 1923, conducted by Major Mark L. Ireland, synopsis of which is to be published by the Advisory Board on Highway Research, discusses in great detail many phases of the problem of energy consumption by highway traffic.

The U. S. Bureau of Standards has carried on an extensive series of tests on this same subject, which it is expected will be published in the near future. It has also been the subject of research at a number of university laboratories; notably those at Yale, the University of Kansas and the University of Michigan.

The subject of highway research has received great impetus and inspiration from the organization of the Advisory Board on Highway Research of the National Research Council. Through this agency, researchers in this field have been brought in contact with work under way and it has greatly stimulated the undertaking of new projects; the field of research has been outlined, and committees organized working in many areas of this domain.

In Bulletin 21 of the National Research Council issued in October, 1922, there are listed 479 highway research projects in the United States. At the third annual meeting of the Advisory Board on Highway Research, held in Washington on Nov. 8-9, 1923, a remarkable set of committee reports was presented. Proceedings of this meeting will shortly appear as a bulletin of the National Research Council. Among the topics discussed in the various papers, included in these proceedings, will be found a number having direct bearing upon concrete highway construction.

LITERATURE ON USE OF CONCRETE IN ROAD CONSTRUCTION.

The literature on the use of concrete for highways would follow naturally the construction, and is of recent date. The only mention made of concrete road construction in the records of the first meeting of this society, Jan. 17 to 19, 1905,[20] other than the use of concrete for sidewalks, was the appointment of W. W. Schouler as Vice-President of a section on streets, sidewalks and floors, but there is no mention of any action by this committee. At the second meeting of the society, Jan. 9 to 12, 1906, George L. Stanley was elected vice-president of the section. His committee reported on salt in sidewalk construction, but there is nothing as to highways or street pavements.[21] Similarly, the report of the third meeting in 1907 [22] contains no reference to concrete pavement construction other than sidewalks and floors, nor is any mention made of concrete highway construction in the minutes of this committee.[23]

In the *Transactions* of the American Society of Civil Engineers, Vol. LIX, December, 1907, Horace Andrews states, "The speaker is of the opinion that, in the severe climates of the northern states, well-laid pavements of inorganic materials are more likely to be satisfactory than others, and he would not overlook the possibility of making an entire pavement, including the wearing surface, of cement concrete. A pavement of this description has given good satisfaction for some years in front of the Capitol at Albany, and seems to be well adapted to automobile traffic."

WM. H. LAWTON.—"Among the monolithic pavements, the principal ones are those composed of portland cement and of asphalts and bitumen. The wearing surface of the former consists of a layer about 2 in. thick, of portland cement concrete, of about equal parts, mixed on the street, and incorporated in place upon the foundation already prepared. The surface is usually blocked off, by deep grooves, into squares of the desired size. This pavement is very slippery, especially in wet or frosty weather.

"All the monolithic pavements deteriorate chiefly by injury from the impact of the calks of the horses' shoes.

"From some form of cement or steel will probably be derived the future pavement."

In the 1907 proceedings of the National Association of Cement Users occurs an article by Albert Moyer on "Cement Sidewalk Paving,"[24] in which he says "Among the other uses (of cement paving) are driveways for vehicles."

It is not until the 1909 proceedings of this society[25] that we find the first paper on concrete highway construction, which was given by J. H. Chubb, which also includes the first proposed standard specification for concrete roads. The discussion of this paper was participated in by W. F. Wiselogel, W. P. Anderson, W. W. Schouler, H. F. Porter and E. V. N. Heermance.

In the meantime, there appeared some articles in the technical press. Among the earliest was a report in *Municipal Engineering*, May, 1905,[26] which was a paper read by H. L. Weber, City Engineer of Richmond, Indiana, before the Ohio Society of Surveyors and Engineers on the subject of concrete paving and paving materials, in which is described concrete alley and street construction first laid in Richmond in 1896. In April, 1907, Mr. Weber as Chief Engineer of the Fort Wayne and Wabash Valley Traction

Company, Fort Wayne, Indiana, and Walter E. Hassam of the Hassam Paving Company, Worcester, Massachusetts, presented papers before the Association of American Portland Cement Manufacturers on the subject of concrete pavement construction.[27] Mr. Weber's paper not only described alley and street construction in Richmond, Indiana, which was laid in the period between 1896 to 1906, but gave also his recommendations for complete detailed specification for a two-course pavement having a 1-in. wearing surface. Mr. Hassam's paper described the type of concrete pavement construction which bears his name.

[28] Since 1909 to the present time, the proceedings of this institute are replete with papers recording the progress and recent developments in concrete road construction, as is also the technical press for the corresponding period.

The reports of the various state highway commissions and state highway engineers contain no mention of the construction of concrete roads prior to 1911 and 1912. In the fourth report of the Illinois Highway Commission, Jan. 1, 1913, there is described the construction of five sections of concrete pavement, including specifications for concrete roads.

The reports of the Maryland Highway Commission mention the construction in 1914 of four sections of concrete highway laid in Dorchester, Montgomery, Somerset and Talbot Counties, respectively, these sections totaling 9.8 miles.

The annual report of the State Highway Commission of New York in 1917 records that a number of contracts for first-class concrete roads were awarded during 1914, and that there had been built by the New York Commission by the end of 1917, 257.46 miles of first-class concrete roads.

The first paper discussing the construction of concrete highways before the Permanent International Association of Road Congresses was presented by the author at the third congress held in London in 1913 and reported in *Engineering News* for Sep. 25, 1913.

The Proceedings of the First National Conference on Concrete Road Building, published in Chicago in February, 1917, and of the Second National Conference on Concrete Road Building, published in Chicago in February, 1916, are notable in a number of respects.

These conferences brought together highway engineers and others directly interested in the construction of concrete highways. Committees were organized a year or more in advance of the conferences covering all phases of the subject. The reports of these committees comprise the proceedings. Never before, in the history of road building, had there been called a conference to study specifically one type of pavement. As a result of these conferences, there are to be found in these two volumes of proceedings a complete summary of the practice, in great detail, as it had been developed up to the time of the holding of the conferences.

During the past ten years, there have appeared many papers and pamphlets which constitute a notable contribution to our knowledge, not alone of highway construction, but, through scientific investigations, of the fundamental physical properties of concrete. These papers are to be found among the proceedings of various engineering and technical societies. Reference has been made only to some of the earlier of these papers. All of the papers which appear among the proceedings of the various engineering and other technical societies are usually to be found summarized in an excellent manner in the engineering press.

REFERENCES AS NOTED BY NUMBER IN THE FOREGOING TEXT.

[1] "Street Pavements and Paving Materials."
Geo W. Tillson, 1900. Page 2.

[2] "Conquest of Peru, Volume 1."
Wm. H. Prescott, 1847. Page 64.

[3] "Vues des Cordilleres," Humboldt. Page 294.

[4] "A Treatise on Roads,"
Sir Henry Parnell, London, 1838.
pp. 10-15 and 151-157.

[5] Special Consular Report issued by Bureau of Statistics,
United States Department of State, 1891.
Streets and Highways in Foreign Countries. Page 326.

[6] "The Construction of Roads and Streets,"
Henry Law and D. Kinnear Clark, London, 1881. Page 243.

[7] "The Construction of Roads and Streets,"
Henry Law and D. Kinnear Clark, London, 1881. Page 162.

[8] "Street Pavements and Paving Materials,"
Geo. W. Tillson, 1900. Page 10.

[9] "Street Pavements and Paving Materials,"
Geo. W. Tillson, 1900. Pp. 140-141.

[10] Association of Portland Cement Manufacturers,
Bulletin 14. Pages 1-17.

[11] Wayne County, Michigan, Highway Commission,
Fifth annual report, 1910.

[12] *Engineering Record*, March 15, 1913.
"Notes on Concrete Road Construction in Wisconsin."
Discussion of paper by H. J. Kuelling regarding use of bank run material and necessity for changing proportions and thereby disorganizing work of crew. Discussion of specification by Geo. H. Randall of Oshkosh, Wis., for 5-in. concrete on ½-in. sand layer on 4-in. 1 : 4 : 8 base.

Engineering Record, March 7, 1914.
"Handling Materials for Concrete Roads."
Committee Report by H. P. Gillette; Don. D. Price, State Engineer, Nebraska; and Percy H. Wilson, A.P.C.M. at National Conference on Concrete Road Building, Chicago, February, 1914.

Engineering Record, April 4, 1914.
"Kinks in Concrete Road Construction."
Paper by C. D. Franks, on construction practices.

Engineering Record, July 25, 1914.
"Industrial Track Equipment for Concrete Road Building."
Discussion of Report of committee at Concrete Road Building Conference, by Louis Hellman, of Orenstein-Arthur Koppel Co. Costs, independence of water, etc., cited.

Engineering Record, January 30, 1915.
"Gasoline Locomotive Hauls Material Trains for Concrete Highway."
Describes method developed by Henry L. Bowlby, State Highway Engineer of Oregon, for this type of haul and handling batch boxes at mixer.

Engineering Record, March 6, 1915.
"Materials for Concrete Road Delivered by Industrial Cars from Central Plant."
Equipment on road between Astoria and Warrenton,—3 miles —by Oregon State Highway Department under direct supervision of Construction Engineer H. L. Burras, reporting to H. L. Bowlby.

[13] *Concrete Highway Magazine*, Vol. 1, 1917.
(Published by the Portland Cement Association, Chicago.)
April, page 18; August, pages 17, 20; September, page 12; October, pages 1, 17, 18, 23; November, pages 22, 23; December, page 22.

[14] *Engineering Record*, October 4, 1913, page 374.

[15] *Engineering News*, April 29, 1915.

[16] National Association of Cement Users, Proceedings. Vol. V, 1909, Page 377 and page 381.

[17] Lewis Institute, Chicago.
Structural Materials Research Laboratory,
Bulletin 1, 1919.
"Design of Concrete Mixtures."

[18] *Engineering Record*, June 28, 1913.
"Chevy Chase Experimental Road."
Features of design described.

[19] California State Automobile Association, Pamphlet 1920.
"How Much Does Your Team Pull."

[20] National Association of Cement Users, Vol. 1, Proceedings 1905.

[21] National Association of Cement Users, Vol. II, Proceedings 1906.

[22] National Association of Cement Users, Vol. III, Proceedings 1907.

[23] National Association of Cement Users, Vol. III, Proceedings 1907.

[24] National Association of Cement Users, Vol. III, Proceedings 1907.

[25] National Association of Cement Users, Vol. V, Proceedings 1909.

[26] Municipal Engineering, May 1905. Page 386.

[27] Association of Portland Cement Manufacturers.
"Bulletin 14." Page 1-17.

[28] This is a list of the more valuable papers to be found among the Proceedings of the National Association of Cement Users:

Vol. VI, 1910.
"Development of Concrete Road Construction."
Richmond, Ind.—Fred R. Charles, City Engineer.
"Use and Cost of Concrete Blocks in Roadway Construction."
Geo. C. Wright, Consulting Engineer, Rochester, N. Y. New York State Highway Commission authorized test on Ridge Road northwest of City of Rochester, N. Y., in which 1800 lin. ft. = 2652 sq. yd. of 2-in. concrete cubes were laid on thin bed laid directly on old macadam roadway, the gravel being at times 2-in. deep and at others no measurable thickness. Filled with fine loamy gravel—except 100 ft., which were filled with portland cement grout.

Vol. VIII, March, 1912.
Revised Specifications of above.
Account of Ann Arbor Practice.
Account of Mason City, Iowa, practice.

Vol. IX, December, 1912.
Standard Specifications.
E. N. Hines on Wayne County, Michigan.
R. C. Stubbs on Vibrolithic.

Vol. X, 1914.

Vol. XI, 1915.

Vol. XII, 1916.

Construction of Toronto-Hamilton Highway by Day Labor.

H. S. VanScoyoc, Chief Engineer.

T. H. Huy, Commissioner.

Coleman Du Pont Road, Delaware.

C. M. Upham, Chief Engineer, Coleman Du Pont Road, Inc.

Proceedings, National Association of Cement Users.

Construction of Easton-Allentown Road.

John T. Gyshart, Jr., formerly Construction Engineer. Pennsylvania State Highway Department.

Essential Features for Successful Construction of Concrete Roads.

Wm. M. Acheson, Division Engineer, New York State Highway Department, Syracuse, N. Y.

Vol. XIII, 1917.

Condition of Wayne County Concrete Roads—A. N. Johnson.

Specifications adopted by letter ballot for

1 course road	No. 5
1 course street	No. 17
2 course street	No. 18
1 course alley	No. 19
Between car tracks	No. 20

Maintenance of Concrete Roads in Connecticut.

W. Leroy Ulrich, State Highway Department of Connecticut.

Some Recent Developments in the Construction of Concrete Roads.

Wm. M. Kinney, Universal Portland Cement Co.

Industrial Haul

Baker

Tractor, Belt, Finishing Machines

Cowell

Stubbs vibrator.

Proceedings, A. C. I., 1922, Vol. XVIII, Cleveland, February 13-16, 1922.

Report of Committee S-6 informally by C. R. Ege discusses present investigations; Pittsburg Test Road, Bates Experimental Road, U. S. Bureau Public Roads at Arlington. Work of D. A. Abrams based on that; similar work by State Highway Departments, showing how we can build good concrete pavements, using such aggregates as are economically available.

Chairman Humphrey.—*Mr. A. P. Davis is well qualified to speak on "Mass Concrete," because of his more than seventeen years in the U. S. Reclamation Service in the development of irrigation projects; in the construction of the dams and other controlling works concrete has been and is largely used,—especially for mass work.*

Mr. Davis was successively Chief Engineer and Director of the Reclamation Service and is a Past-President of the American Society of Civil Engineers; he is unable to be here because of professional engagements in Washington. But that misfortune is a blessing in that it affords us the opportunity to have before us a fellow member who has been very active—giving a great deal of energy and of time to the assembling and arranging of the excellent and very interesting historical exhibits which form a memorable part of this Anniversary. He needs no introduction to you.

It is my pleasure to present to you Mr. A. W. Stephens, who is not only a good fellow, but a very good engineer and an expert in reinforced-concrete design and construction, who will read Mr. Davis' paper.

Use of Concrete for Dams, Tunnels, Culverts and Canals.

By Arthur P. Davis

Consulting Engineer, Berkeley, Calif.

The subject assigned this paper *itself* illustrates one of the greatest merits of concrete, namely, its adaptability to a large variety of conditions. The use of concrete is such an advance in masonry construction as was made in the use of iron when it became practicable to melt and cast it in the form desired.

The manufacture of artificial stone in the form and position desired is an art that has advanced greatly within the past few years, and is still doubtless susceptible of improvement. It is possible to form such stone of good quality by depositing it deep in water, or to blow it through pipes for tunnel lining or in similar locations difficult of access. Such facility of manipulation lends itself especially to the requirements of tunnels, conduits, culverts and other structures involving complicated details and accessories.

Though the advantages of concrete over natural stone are important as regards convenience, speed and economy, yet probably

its greatest advantage is the facility with which metal reinforcement can be incorporated to any extent and in any form desired, so that it can be made monolithic, and can thus preserve complicated forms and shapes, with such distribution of strength by appropriate distribution of reinforcement as the needs of the structure may require. This valuable quality makes concrete adaptable with facility and economy to a great variety of structures which

SUN RIVER PROJECT, MONTANA.
Fort Shaw Unit, Simms Creek pressure pipe looking west.

are entirely outside of the realm of natural stone, or indeed of any material approaching concrete in permanence and durability.

The adaptability of concrete to the requirements of the complicated and elaborate structures alluded to above, depends to a large extent on the practicability of providing and manipulating the forms necessary for such purposes. In fact, cases occur where the cost of forming and of manipulating the forms render the substitution of some other material desirable.

The developments in this line have made notable advance in recent years, and we may expect improvements in the same direc-

tion in the near future that will add materially to the use of concrete in structures of the kind mentioned.

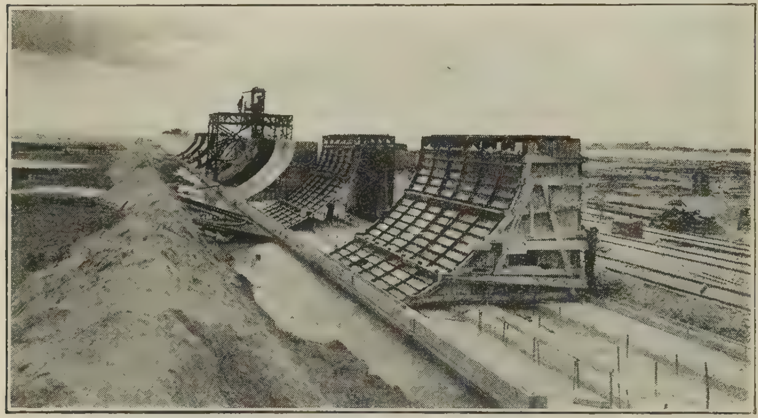
Concrete is essentially artificial stone. Its manufacture involves a chemical process, requiring a length of time which affords opportunity for handling and depositing it in molds or such other positions as may be desirable. It is necessary to complete any necessary manipulation of the concrete prior to the time that the hardening begins and hence before the chemical action has ma-

ELEPHANT BUTTE DAM UNDER CONSTRUCTION, RIO GRANDE, NEW MEXICO.
Showing Columnar Construction Method.

terially advanced. It follows that practically all of that chemical action occurs after the concrete is in place. This chemical action generates an appreciable amount of heat, which in turn increases the volume of the concrete while it is still in a condition to adapt its shape and volume to its environment. As the hardening proceeds under the increased temperature, its shape becomes permanent, and when the chemical action is completed the temperature subsides, and the volume is reduced accordingly. As the concrete contracts on cooling there is a tendency for long structures, such as pavements, walls, etc., to crack at intervals, which vary with climatic and many other conditions, but which must be reckoned with,

if such cracks are detrimental, or for any reason undesirable. If such cracks are left to chance, they are ragged and irregular, and as changes of temperature occur, incipient disintegration is encouraged, and hence it is desirable to control the form and position of such cracks, by the provision of contraction joints, which can take care of the small movements induced by temperature, and which are of many kinds, varying with the conditions that must be met.

Some recent experience shows that the tendency to form cracks or to open joints provided, is less if the concrete is placed



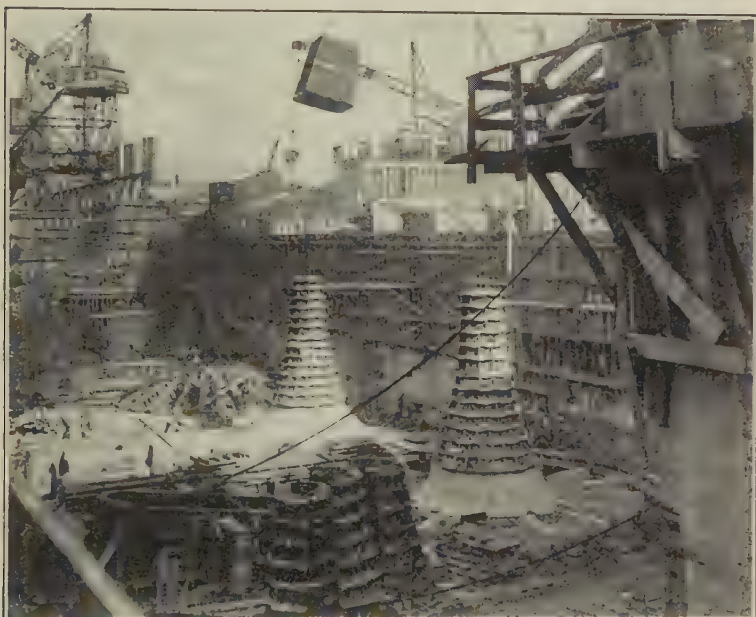
GALVESTON SEAWALL EXTENSION.
Steel Superstructure Forms with Form Traveler.

in cold weather, or as to place it in compression when warmer temperature arrives, and minimize the contraction in cold weather. These problems assume great importance in conduits, dams, and similar structures where leakage is undesirable. In the case of concrete pipes, not only is it desirable to place the concrete under conditions of minimum temperature, but to restrict changes of temperature by burying the pipe. By following these rules, concrete pipes properly reinforced have been made to successfully conduct water under pressure heads exceeding 100 feet.

This has been accomplished by first precasting the pipe in convenient lengths in a central yard, where by suitable appliances and proper materials first class work could be done, and the curing

of the concrete carried out in a proper manner under favorable conditions. These pipes were laid after all the chemical heat of setting was dissipated, in weather colder than the mean annual temperature of the locality, the joints were carefully sealed, and the pipe properly covered to a depth sufficient to protect it from great changes of temperature.

The success of past experience indicates that reinforced concrete can be advantageously employed for pressure pipes involving



WILSON DAM, TENNESSEE RIVER.

Dec. 17, 1923—Draft Tube Cones at North End Power House.

much higher heads than any yet attempted. Progress in this direction may be expected in the near future, and is rendered very desirable by the unstable and temporary character of such other substances as are adaptable to the uses described.

The problem of expansion and contraction has been met in the cases of at least two high dams, in a manner that may be worthy of description.

One of these dams was straight, and the other curved on a long radius, i. e. (785 ft.), and both were designed to be stable as

gravity structures. In order to forestall and prevent the opening of cracks in these dams, they were built in columns, separated by joints normal to the faces of the structure and alternate columns were built to considerable height, while those lying between were left a considerable time for later construction. After the columns first constructed had completely hardened and lost their chemical heat, and so far as practicable in the latter part of the winter, while the cold of winter had chilled the entire body of the columns built, and reduced them to a minimum volume, then the other columns were built. The latter were of materially smaller volume than the former, so that the chemical heat and resulting expansion would be correspondingly small.

By this proceeding, the whole structure is placed in compression during the heat of summer, and the results show that the joints do not materially open when winter contracts the volume of the masonry. In addition to preventing leakage the condition of high compression thus induced increases the tendency in the case of the straight dam, to act as a beam, and in the case of the curved dam, to act as an arch. At any rate the precautions taken have resulted satisfactorily, and have added nothing material to the cost of the structures involved.

Somewhat similar precautions have been taken with concrete lining for conduits, and where this is carefully done, the precautions have been justified by the results.

Since many of the structures of which this paper treats, require a high degree of water-tightness, experiments on this point have been from time to time carried out, and advance has been made along this line. One of the most efficient measures yet used is the application of the cement gun. This machine properly used produces a stone, so dense, hard and impervious as fairly to be considered in the first rank of its kind, and a plaster so made and applied can be made to adhere to old concrete or other structural materials, in a degree highly satisfactory and useful.

The transportation and placement of concrete by the pneumatic method, especially in positions difficult of access otherwise, such as in tunnel linings, has shown such economy and excellence of results as to encourage further experiment and to promise important results.

The transportation and deposit of concrete by pouring through pipes and troughs, which has rapidly grown in favor in recent years, has received something of a jar by the experimental demonstration of the serious dangers of using too much water in the mix, and further study is needed to ascertain the feasibility and means of reconciling the chemical needs of the mix, with the fluidity necessary for rapid and economical placement, and satisfactory moulding against the forms. The need of further information on this line applies as well to the pneumatic method of transportation and deposit.

The American Society of Civil Engineers has appointed a committee to examine the movements and the stresses in arch dams, and as these are nearly all built of concrete, we may expect from this committee a valuable contribution to the literature treating of the use of concrete for this purpose.

Another encouraging feature of the present outlook is the tendency to discuss, and encourage discussion of the various uses of concrete, and experience therewith, which is exemplified by this and similar meetings.

CHAIRMAN HUMPHREY.—*In the history of the Cement Industry in this country there is an outstanding personality—an important factor in its development—a pioneer in the art—a man of fine character and of indomitable perseverance. The industry owes a great deal to the late Ernest L. Ransome for his labors in developing the art of Reinforced-Concrete Construction.*

His forebear was Frederick Ransome, the inventor of the Rotary Kiln, which has had such a marked influence on the development of the Portland Cement Industry. It seems particularly fitting that another Ransome, also in the industry, the son of Ernest L. Ransome, should address the Institute on

"The Mechanical Equipment for Handling Concrete"

it is my pleasure to present to you

MR. A. W. RANSOME.

THE MECHANICAL EQUIPMENT FOR HANDLING CONCRETE.

BY A. W. RANSOME

OF RANSOME & MCCLELLAND, INC., SAN FRANCISCO, CALIF.

DURING a single life-time concrete has progressed from a position comparatively unimportant to its present pre-eminence in the field of engineering construction. This is in large part due to the development of efficient tools to carry out the performance of the work.

As a small boy I played in a yard among fragments which marked some of the earliest attempts to make concrete something more than a material from which to make sidewalks, to cast lintels, flower pots or baptismal founts. It is rather an astonishing fact that in that yard were conceived most of the fundamental ideas underlying both the construction and the mechanical sides of the industry. To confine myself to the mechanical side, we have the present day mixer, the power loader, the flat top measuring barrow, the concrete hoist bucket, the two-wheel cart, the boom and bucket distributor, all these ideas were conceived there, as witnessed by the files of the U. S. Patent Office.

In the beginning, the user of concrete was faced with the problem of how to mix the ingredients together into a plastic mass and

how to transport this mass to its final resting place in the elevated forms, or into a hole in the ground, as the case might be, where this plastic mass was to harden and become an integral part of the structure desired. No mechanical equipment was existent for this work of mixing, hoisting and distributing the plastic mass and it was soon evident that the strides that were being made by the pioneers in design and use of concrete and reinforced concrete, and their research as to its adaptability to structures of ever-increasing magnitude and importancee, would be of no avail, if economical mechanical means could not be found to replace manual labor in the actual manufacture of concrete. Without mechanical aid for the work, the scope of the industry must be forever limited, due to the number of man hours involved in mixing and transporting concrete without mechanical aid. In this industry, to an unusual degree, we find the demand for increased output, combined with the necessity for reduction of the labor involved, as the two factors exerting a resistless pressure for mechanical aid. This demand has been answered effectively by the genius of the equipment engineer so that today, on any great project constructed of concrete, either plain or reinforced, we see an array of powerful and efficient machinery all designed and built for its specific duty, giving an almost unlimited capacity for mixing and placing concrete and reducing to a minute fraction the man hours that would have been originally involved in the same operations, had such even been possible without the aid of machinery. The adoption of the present day equipment for handling concrete has had another equally valuable influence on the development of the art in that the inequalities in concrete are largely removed and now, with the uniform gauging of materials, thorough mixing and rapid distribution, the designing engineer is relieved of the great worry as to the actual strength of the concrete as it is placed in the forms, and with this contingency removed, he is justified in practicing economies in design that would be unsafe, were he not sure of the quality of concrete.

The materials were spread on a platform of wood or steel, in layers, one on top the other, measured in bottomless boxes. They were turned three times dry and three times wet, the third turn often being into the work or into the barrow which conveyed it to the work. We shall see this dry mix persisting for a considerable

period after machine mixing came into use; likewise we shall see persist the tendency to short cut operations involving fatigue.

Those were the days of the Irish, Swede and German laborer and we considered it a fair day's work for a crew of eight men to mix and place on the level 40 cu. yd. per day, or 5 cu. yd. per man. Present day labor will not average more than 2 yards per man. We pay the 2 yd. man 60c per hour; we paid the 5 yd. man 15 to 17c per hour.

To turn out such a yardage as is frequently required today, would necessitate a return to labor conditions prevailing in the time of the Pharaohs, if we attempted to mix by hand. On the Hetch Hetchy Dam six men and two mixers turned out 160 cu. yd. in an hour; mixing by hand would require 640 men and an acre or two of ground. Not least among the junctions of plant is that of keeping the organization within reasonable limits.

The first mixer was developed more because of a desire to improve the quality of the mix than to reduce the actual cost of mixing.

In the days of baptismal fonts and flower pots, Mr. Ransome had used a mixer which was essentially a barrel with a door at the base. A vertical shaft fitted with paddles was driven through bevel gears. The top was open.

The demand was for "dry mixed concrete," etc., "to be deposited in layers not exceeding 6 in. thick, and rammed until moisture collects upon the surface." The tampers in those days weighed 14 to 20 lb., had faces 6 to 12 in. square, and were mankillers. It was a curious thing how the handles of those tampers were always highly polished. The men had a habit of resting the tamper and moving the hands only.

Mixer No. 1 was quickly followed by No. 2, which was in effect simply a variation of the Chilean Wheel. This in turn was followed by a long 30 ft. octagonal drum, also of wood. This was set on an incline, and the materials fed into the drum passed through continuously. It was necessary to measure the materials on a platform in layers.

This machine was followed immediately by a wooden cube with a hollow trunnion through which water was fed to the batch. I might say here that in all the early mixers it was considered necessary to mix materials first dry, and the water was added later. The

introduction of the hollow trunnion was a great time saver in that it was not necessary to top the cube for the water. The introduction of this hollow trunnion was an important step.

It was a simple matter now to enlarge this trunnion to the point that all materials could be fed through the head, and the mixer revolved continuously.

The next step was the adoption of the cylindrical drum and in 1885 appeared the prototype of the present Ransome mixer. This first mixer was built of wood, with movable blades which were set in mixing or discharging position through cams and quadrant. A folding chute extended through the drum. The discharge required two operations, and required approximately 35 seconds for a half-yard batch.

In 1900 the movable blades were eliminated and the folding chute was discarded, resulting in the discharge of a batch in 15 to 18 seconds. It is at this point, the discharge, that the greatest savings have been made in the mixing operations. The Ransome mixer of today will discharge a full one-yard batch within 12 seconds.

Since 1900 the development has been mainly in the direction of higher mechanical efficiency of the parts. Attention has been focused mainly upon the refinement of details rather than radical changes in design.

Ten years ago I guaranteed a Ransome two-yard mixer to handle 50,000 cu. yd. without repair. On the Hetch Hetchy project two Ransome mixers handled each 200,000 cu. yd. without repair other than one renewal of blades. Blades have been increased in thickness, and it is only a matter of time until a special alloy steel will be substituted for the present material.

Hyatt roller bearings were repeatedly tried out and found wanting when used in connection with mild steel. To get results a special high carbon shaft is essential.

We struggled for several years with the problem of securing traction rings on the drums that would not wear cuppy. Rolled steel tires were shrunk on cast iron rings in the search for effective wearing surfaces and finally came the standard locomotive tires, which are standard construction on the present Ransome mixers. We found that one of the contributing causes to uneven wear on the traction rings lay in the practice of keying both rollers fast to

the roller shaft. The action of these rollers is similar to the action of the wheels on the ordinary trolley car. It was the sight of a crew of men on the corner of Jackson and Dearborn streets grinding down the humps out of trolley tracks that lead us to the conclusion that this practice of two rollers keyed on a single shaft was wrong, and we adopted rollers fitted with Hyatt roller bearings revolving free on high carbon shafts.

The period from 1898 to 1910 saw a great many machines of radically different principle tried out for mixing the ingredients of concrete, but each in turn has been discarded for what is known as the batch mixer. Government engineers in the Reclamation Service constructed what was known as a gravity mixer which was nothing but a series of hoppers, one under the other, the ingredients being dumped into the top hopper and then allowed to fall by gravity through the series of hoppers, the necessary water being supplied to the dry ingredients, as they fell from one hopper to the other by gravity. The results of this scheme was not dependable, as the heavy aggregate was more apt to separate from the finer aggregate than to mix with it. A further development of this first principle of gravity mixing was a vertical tunnel in which the ingredients of the concrete were deflected from side to side by baffle plates. The results of this scheme were not any more satisfactory than that given by the series of hoppers first used. At a later period a good deal of attention was devoted of what might be called a continuous type of mixer which generally took the form of a horizontal trough wherein the materials were charged at one end and discharged at the other, through the action of spades attached to a shaft, rotating along the axis of the trough, the water being supplied to the dry ingredients from a perforated pipe over the top of the trough. The mixing in this type of machine was faulty and the proportioning of the ingredients even less satisfactory, as the sand and cement was usually shovelled in at one side of the trough and the stone in the other, the relative proportions of the sand, stone and cement depending entirely upon the speed with which these materials were shovelled into the trough. Various automatic devices were also tried.

While in appearance the mixers manufactured by a considerable number of firms vary a great deal, it is truthful to say that they are all operated on the same general principle. A batch of

concrete composed of definite proportions of ingredients is placed within rotating drums of various shapes and with various kinds of baffle plates or blades and is tumbled in these drums until it is thoroughly mixed and is then discharged before another batch of ingredients is allowed to enter the drums. The capacities of these mixers vary as a rule, from a small one-bag machine which will turn out ordinarily about one-sixth of a cubic yard of mixed concrete per batch, to the huge machines which will turn out two yards of mixed concrete at a time. The smaller machines are used where small amounts of concrete are desired at different points and their size, as a rule, makes them easily portable, so that the machine itself may be moved around, rather than make necessary the transportation of the mixed concrete. The large machines, we generally find set up in a permanent location, and it is found economical to construct storage bins fed by conveyors or derricks so that the capacity of the big machines may not be decreased by delays in charging the same with the dry materials and water. In cases like these, elaborate arrangements are usually made for transporting the mixed concrete to its final resting place, inasmuch as the magnitude of the big mixer installation represents too much of an investment for the same to be pulled down and moved as the work progresses away from the same.

During the construction of the Panama Canal, mixers of 4 cu. yd. capacity were designed for some of the lock work, but while they worked successfully, they were of such a cumbersome nature that it is probable that if that work were being done today, we would find batteries of smaller mixers, probably of not more than two yards capacity, replacing the fewer number of gigantic four-yard mixers. In the ordinary run of concrete construction work, especially as applied to buildings, a machine of one-yard capacity is about as large as is ordinarily used. It has been found possible with such a machine, when the time of charging and discharging is cut to a minimum by the use of storage bins from which the ingredients are charged into the hopper by gravity, to produce a batch of concrete at intervals of a minute and a half and yet allow a satisfactory period for mixing within the machine. Such a machine has, therefore, a capacity of over 300 cu. yd. of concrete in an eight-hour working day and this capacity is usually sufficient for the area that can be economically handled by one machine.

At about the same time that developments and refinements were taking place in the mixer, the need of a more economical method of hoisting concrete was obvious. When the first mixers were used, the concrete that was to be elevated was generally dumped from the mixer into a side or bottom dump bucket which had been run under the discharge chute of the mixer on a small flat car. When loaded, this bucket could be run back to where it could be reached by the fall of a derrick and could then be hoisted by this derrick and the contents of the bucket deposited in an elevated hopper. From this hopper the concrete could be distributed by wheelbarrows or possibly the bucket could be set upon another flat car at the higher elevation so that the whole might be moved over a system of industrial tracks to the location where concrete was being deposited. On small work, the concrete was usually drawn directly from the discharge chute of the mixer into wheelbarrows which were then run on a platform hoist such as is generally constructed for handling brick work. The first great improvement in the methods of hoisting concrete was the development of a special tower with guides in which some form of a concrete bucket was housed, this bucket automatically discharging its contents at the proper elevation into a concrete hopper from which the concrete was in turn discharged into carts or wheelbarrows, the flow being controlled by gates. This system of hoisting concrete has not changed very much from the time of its first adoption, the principal changes being rather in the refinement and improvement of the mechanical features of the operation. The details of the buckets have been greatly improved so that they may discharge more rapidly and with less jar to the hoisting tower. The details of construction of the buckets have been improved so that the cost of maintenance is much less than in the beginning. The first towers were all built of wood, the timber members being of various sizes, depending upon the height of the tower and the capacity of the plant, the method of vertical posts and cross bracing differing considerably on each installation.

There was practically no salvage to a wooden hoist tower after the job was completed, although a great deal of time and money may have been spent in the proper fabrication of that tower. Many builders realized the waste due to the expensive framing and building of a tower and then junking the same at the end of the opera-

tion. This thought led to the development of the steel hoist tower which could be easily erected and taken down and used an indefinite number of times, with only slight repairs. We now see the steel hoist tower in its various forms on the majority of the larger operations and it has become a regular item of plant in the yards of most of the large builders, who specialize in concrete construction work.

With the development of the mixer and the hoisting plant so that large quantities of concrete could be mixed and hoisted with rapidity to any elevation desired, it was soon noticeable that the principal congestion would take place in the lateral distribution of the concrete to the points where it was to be deposited. At first, wheelbarrows were used and with their capacity of only a few cubic feet, a great number of wheelers was necessary to carry away from a powerful concrete plant the output of the same. The ability of a single man to distribute concrete was greatly improved when the two wheeled concrete cart was devised. My father was the inventor for both the present day hoisting bucket and the two-wheel carts. With this means of transport, a wórkman could handle five to six cubic feet of concrete with less effort than he had formerly handled two feet. It still seemed, however, that too many man hours were being consumed in the lateral transportation of concrete from the hoist tower to the place of deposit and this led to the development of the transportation of concrete through inclined chutes. Experiment showed that concrete, in a plastic condition. could be easily transported through metal chutes of proper shape, when these were inclined at an angle of about 23 deg. to the horizontal. The transportation of concrete through chutes simply meant that the height to which the concrete was elevated had to be increased slightly and then a wide lateral distribution of concrete could be made by gravity. It is probable that more time has been devoted by plant engineers to the development of chuting systems than to any other item of plant and today we find many refinements in that equipment, especially as to its general efficiency and the ease with which the point of discharge may be moved around.

One of the most successful details that has been worked out in the refinement of the equipment for chuting concrete has been the development of what is known as a sliding frame attach-

ment for the concrete tower, which, when used in conjunction with a counterweight boom equipment, gives a very mobile plant that can be easily elevated from floor to floor and one that has great elasticity for lateral movement. This kind of equipment is especially adapted to the construction of buildings where only relatively small quantities of concrete can be deposited in one place without the lateral or vertical movement of the discharge chute. For heavy dam work and work of a similar character, we still see heavy straight line chutes supported on cable ways, inasmuch as this type of chute can be used for the widest distribution of concrete, but naturally it is only suitable where large quantities of the material are placed in certain locations, due to the difficulty of changes in alignment of such equipment.

In the same manner that very small mixers have been efficiently developed for very small work, so hoisting plants of small capacity, but of extreme simplicity and ease of erection, have been developed, where the quantity of concrete to be elevated is so small that the expense of erecting an ordinary concrete tower could not be borne. Such equipment is known as a mast hoist. wherein a timber or steel mast is erected to a considerable height. An especially made bucket which takes the discharge of concrete directly from a mixer, slides upon this mast and discharges its output into a smaller hopper, also supported by the mast. This hopper is easily raised and lowered and from this hopper the concrete is carried to point of deposit by small steel chutes. Such equipment is found economical for hoisting concrete where the quantities to be placed per day are small.

For special work, such as the lining of tunnels and other underground concrete work a system of pneumatic transportation has been devised and operated with some success. It is probable, though, that such transportation of plastic concrete will never come into general practice. For long distance carry by this method, it is difficult to keep uniform the velocity of the transporting air and the heavier parts of the aggregate have a tendency to lag behind the lighter parts, often necessitating a re-mixing of the concrete when discharged. Difficulty has been found in handling the pneumatic method of transportation of concrete through pipes, because where this transportation takes place over any great distance, there is always a very considerable quantity of concrete in transport and

in case of a necessary shutdown, this concrete has to be cleared from the transporting pipes and probably wasted. In case of any breakdown in the system, great danger arises from the possibility that the concrete cannot be removed from the transporting pipes before its initial set clogs up the system of transportation.

Space is not allowed here to even mention the numerable appliances that have been developed to increase the efficiency of the mechanical handling of concrete, but it is safe to say that at the present time the engineer laying out the plant to handle the concrete work on any project finds at his disposal such an array of efficient machinery for every phase of the work that he is really apt to use too much plant, instead of too little.

It is one of the problems of the engineer having charge of this part of the work to make a careful analysis of what he has to do, so that the value of the depreciation of his plant and the cost of its installation will not exceed the value of the labor to be saved by its use. This phase of the problem has been discussed in the proceedings of your Institute during the last several years by the report of the committee on contractor's plant.

CHAIRMAN HUMPHREY.—*In these days when reinforced-concrete is of such general application perhaps few of us have a real appreciation of just what the embedment of metal in concrete has done in opening up a great industry. The reinforced-concrete work we are doing now is primitive as compared with what may be attained—we do not at present think it is primitive, but it will be so considered, the speaker is sure, in the light of the knowledge and achievement of the years to come.*

In the days of ancient Rome, when men were beginning to get some idea of the value of cement, architecture covered engineering; and so Vetruvius, when he wrote his work on "Architecture," was also speaking of the engineering of that period. It seems particularly appropriate therefore that in opening the afternoon's session on concrete the first speaker should be an architect. This particular architect is a man who has had vision and has been able to apply concrete without any indication that he had to use it, but with appreciation of its real merits; it is with pleasure that I present Mr. Albert Kahn, who will speak on

"Reinforced-Concrete Buildings."

REINFORCED-CONCRETE ARCHITECTURE THESE PAST TWENTY YEARS.

BY ALBERT KAHN

ARCHITECT, DETROIT, MICH.

THIS, the twentieth anniversary of the American Concrete Institute, is a very proper occasion for reviewing what has been accomplished these past years in the field of reinforced-concrete.

Considerable has been done in the comparatively short time. Unexpected results have been attained, progress has been constant, but much still remains to be done and undreamed of possibilities will be realities within the life-time of many present.

Naturally, the early history is full of experiments. That such systems as were exploited at the outset by metal lath, wire fence and cable manufacturers did not result in more failures is inexplicable and miraculous. When we recall the numerous structures erected at the time with floors of 20 ft. and 30 ft. spans of cinder concrete reinforced with cables stretched from wall to wall, the

structural design mere guess work, we can only marvel at the protection of a kind Providence and hope that the interior partitions in these buildings may never be removed. Many indeed were the fools who rushed in where angels feared to tread. But fools, men with vision, if you please, are necessary in the progress of mankind and were needed in the development of reinforced concrete. Of course, there were failures, serious ones, but upon these failures were built the successes that have followed.

We recall, as with all things new, the serious opposition encountered at the outset from laymen who could not comprehend how a wet mixture of sand, stone and portland cement with a certain number of steel rods placed about could carry its own dead weight, much less heavy live loads. But even more strenuous was the reaction from engineers and architects. Indeed, the chief difficulty was to overcome the resistance and inertia of these professional men who had feelings of doubt and clung tenaciously to the forms of construction with which they were more familiar. Somewhat like the country man on a visit to the city who entering a building asked the woman in charge, "Be this the Woman's Exchange?" "Yes," was the reply. "Be you the woman?" "I am." "Well, I guess I'll hang on to Maggie."

We remember the adverse efforts of the clay tile industry, which, with the introduction and development of the new construction feared ruin for itself. Who does not recall their trade journals with glaring accounts of concrete failures and articles predicting the direst happenings?

The structural steel interests were no less strenuous in their vilification. They, also, feared heavy inroads upon their product, groundlessly, of course.

Even the lumber interests combated the advancement of the new construction, but they now find that their material is required in greater quantities than ever, the lumber necessary for form work being no small factor in this increased demand.

A very serious obstacle to the development of reinforced-concrete was the over-enthusiasm of its early devotees, who, believing it a panacea, undertook to use it for any and every problem. We have since learned that isolated columns and beams are cheaper and quicker of erection in structural steel fire-proofed, and that long span trusses high up are as a rule more costly and more dif-

ficult to construct in reinforced-concrete than in structural steel. Some ill-advised undertakings naturally had a retarding effect upon the progress of the art, but they were necessary to determine the limitations of the new construction. Today the practical engineer or architect quickly decides which best serves its specific purpose and does not hesitate to use the different materials in combination, to the end of producing the best result, most economically.

Another hindrance was the lack of authentic data on reinforced-concrete. Everybody devised his own method of computation and needless to say no two agreed. As to the methods of constructing form work and when to strike same, all was mere guess work and many a mishap was caused by the lack of definite knowledge or information. Out of this chaos there developed a few men who by means of tests and experiments contributed their ideas and results for safe formulae and methods. Monier, Coignet, Considere and Hennebique gave reinforced-concrete its real start. Its present highly developed state in this country, however, is due in the largest measure to engineers connected with commercial institutions who possessed the courage and initiative to design and build structures in the more practical methods conceived by them. They were real pioneers, did the real work, made the mistakes and suffered the consequences. Their errors and resultant losses formed the foundation upon which the theory and practice of reinforced-concrete was really established.

Chief among the objections at first to reinforced-concrete was its dead weight and consequent size of the columns. Thereupon were developed the hooped column and the various types of hollow floors.

The introduction of the flat slab system created great consternation in the minds of architects and engineers and met with the most determined opposition. How was it to be calculated? It stood up, but how and why? Methods of computation had to be revised and the dependence upon tests was increased. Whatever the extravagant claims made for this type of construction by its first advocate, it is one of the important contributions of America

History is still in the making. There will be further developments in practice, building laws will be revised to take fuller advantage of the potentialities of reinforced-concrete, concrete design will grow lighter, more elastic and more economical. Greater

knowledge of the subject, better materials, more perfect control and the concerted efforts of this association will help to bring this about.

But enough of this. You gentlemen are familiar with the history of reinforced-concrete in building, wherefore I shall address myself more to what architects and architecture have contributed. As was to be expected, reinforced-concrete in the first instances in this country was employed merely in the construction of skeleton frames, much as in steel, with the one difference, that the floors were part and parcel of the frame, which latter made for a more rigid building. The exterior facings were masonry of one kind or another supported by and enclosing this frame.

European architects less familiar with this method of building, employed concrete from the outset for exterior facings. With labor costs much lower, and careful workmanship more general than here, it was only natural that they should produce results quite impossible in this country. Furthermore, the climate in those parts where reinforced-concrete was most used was less severe than in most sections here. They, therefore, had a different problem.

Concrete being plastic in its initial state, it was easy to mould it into forms, which were quite impossible with stone or brick, the very facility with which the material could be shaped proving the prime cause of much that is bizzare. The "Nouveau Art" movement was sweeping Europe just at the time and exerted a deleterious influence upon design in general and particularly reinforced-concrete which permitted the use of such extraordinary forms. The wildest conceits were foisted upon the public as works of art. Nor has the orgy ended, for "Nouveau Art" has since given way to still more degenerate forms attending the latest and worst of all, the "Ultra Modern" movement. What this means is proved by examination of most of the foreign designs submitted in the competition for the *Chicago Tribune* Building, many of which are mere travesties. At that, much of the pre-war work of especially German architects is outstandingly good.

In France, the use of reinforced-concrete grew rapidly. It lent itself especially to the extravagances in which its modern architecture loves to indulge. Whatever the opinion of French architects of their productions, it is well that at least American

architects have allowed themselves to be but little influenced thereby.

In Belgium, Italy, and Austria, the French precepts were largely followed, with results even more flamboyant and queer. The sanest, and certainly the most comprehensive development has been that in the United States. Here the material was first used for purely utilitarian purposes as a means to an end, the latter being fireproof buildings, well lighted at low cost. The new system of construction seemed particularly adapted for industrial buildings. The skeleton frame of a material that, unlike steel, required no further covering for protection against weather afforded a maximum glass area so essential in manufacturing buildings; the scheme of slender columns, curtain walls and large surfaces of glass, the rigid vibration-proof, fireproof contruction, at a price only slightly more than that of mill, quickly appealed to manufacturers and grew in favor in spite of the early opposition. Even the first efforts without attempt at design produced more attractive results than were common with the old-style factory buildings. Since then there has been distinct advance and there has been created something akin to an individual and characteristic type. Our successful results in the main are founded on the age-old principle of employing forms suitable to the material and avoiding forms difficult to execute. Simplicity, honest recognition of what was possible with the new material, direct and frank expression of the problems have been the keynote.

At first, there were attempts at imitating other materials, particularly by blocking and forming rustications, complex belt courses and cornices, to imitate stone. But use of forms characteristic of other materials and imitation have always proved a failure. In concrete it could be no different. On the other hand, frank appreciation of the limitations, adherence to well-established laws of composition and proportion, have ever been underlying principles of good design. Indeed the very restrictions have often proved the basis of successful work.

Since a reinforced-concrete structure is primarily a series of columns, supporting lintels and floors, the most direct external expression would emphasize the vertical element. In most of the best work, therefore, the vertical line predominates, piers of varying sizes extending from the ground straight up, even past the roof

line. Good proportion in mass as well as solids and voids, play of light and shade produced by judiciously managed projecting surfaces, restrained use of mouldings and ornamentation are characteristic of the best concrete work. And so effective have been the results procured by observance of these restrictions, that they have been applied to other modern masonry architecture with distinct gain in appearance.

Architecture has ever been the recorder of man's progress. This is an age of utilitarianism, of the practical, and today's architecture is a manifestation thereof. We do not lack appreciation of the beautiful and there exists the desire to encourage art—but only to the degree that it will not interfere with the chief object, namely—the serving of a particular purpose. Such an aim would hardly seem conducive to the greatest art and yet, through the efforts of men equal to the task, buildings of architectural excellence, though meeting the day's requirements are not uncommon.

Now let us refer to specific examples calling attention as far as possible to distinguishing characteristics. As already stated, the first reinforced-concrete buildings in this country were those constructed for industrial purposes, as a rule a skeleton frame of exposed concrete with voids between columns filled with glass, often extending from floor to ceiling without even panel walls. There is little of architecture in such a structure. A slight attempt at design was introduced by providing the building with some sort of cornice. As before mentioned, other materials were imitated at first, with unfortunate results. Then followed franker acknowledgment of the material, with no attempt at making concrete look like stone. This was an improvement. With the feeling that exposed concrete was unattractive, uninteresting in color and texture, the use of brick and other colorful materials in conjunction with concrete was introduced. Then with the consideration of the building in mass and to relieve the monotony of uniformly spaced columns came the corner pylons to form a proper abutment, occasionally in long fronts, also intermediate pylons to vary the design. Concrete panel walls gave way to such built in brick, often entire portions of the frame were veneered in brick, using the exposed concrete as trimming. There is nothing wrong in such a combination, even though it be camouflage, aesthetic laws to the contrary notwithstanding. Laws in architecture are

BROOKLYN ARMY BASE.
Cass Gilbert Architect

much like the eighteenth amendment. They exist often only to be broken, with this one difference, however. Architectural laws can be changed—the amendment, I fear not. All depends on how combinations are used, the expedient considered impossible by the mediocre, is often, in the hands of the expert, the very element of outstanding results.

On the other hand, much excellent work has been done entirely in concrete. Take Mr. Gilbert's Brooklyn Army Supply Base Building. Nothing could be finer in mass, more straightforward or more direct. It is almost bald in its simplicity, certainly ornamentation plays no part in the design and yet by a masterly vertical sub-division of pylons, piers and mullions, all in proper relation to each other, all splendidly proportioned not only in width and height, but also in relative projection, we have a work of the highest architectural merit. Nor has the architect tried to conceal the character of the material. Indeed he frankly accepts it and makes a virtue of what would seem a blemish. He makes no attempt at hiding even the board marks of the forms. Instead he procures a certain texture thereby. Throughout there is revealed a fine appreciation of the material employed and the problem in hand. It is truly the work of an artist and incidentally proof positive that even the plainest structure may be made attractive; furthermore, that the architect is a much needed individual in spite of the opposite opinion of many engineers. Compare Mr. Gilbert's building with another reinforced-concrete structure in Brooklyn, one built for the Navy and designed by its own engineers and note the difference.

Another important group for the Navy is its office building in Washington. It is inoffensive but it fails in that its design does not express concrete. It might as well be constructed of brick or stone. Mr. Gilbert's is a concrete structure and could be mistaken for nothing else.

A later example of much interest is the Fletcher Building on Varick street in New York, by Helmle & Corbett. Though not as characteristic of the material as Mr. Gilbert's Army Supply Building, it is yet an important contribution and shows the potentialities of concrete design. There is here an interesting grouping of parts, the vertical treatment emphasized, the base and top properly held together and the corners reinforced, all producing an excel-

VARICK STREET BUILDING, NEW YORK.
Helmle & Corbett, Architects.

lent architectural composition, without sacrifice of practical needs. If there be any criticism, it might be that the belt course, finals and other details savor of stone design. But the building is so good as to outweigh such minor objections. Incidentally, there has been in this building an attempt at surface treatment which subject I shall touch upon later.

The Marlborough-Blenheim in Atlantic City is one of the earliest of important reinforced structures in this country, and though its enclosing walls are of hollow tile stuccoed, the exterior design is typically concrete. It was indeed a courageous undertaking. Architecturally, this building ranks high. There is a playfulness and picturesqueness which are well suited for a resort of the kind. Then, too, there is much originality in the design without being outré. Price & McLannahan were its architects.

The Hide & Leather Building by Thompson & Binger is a skyscraper built of reinforced-concrete. Just what are the practical limitations as to height of reinforced-concrete buildings is a matter of opinion. The sizes required for columns are usually the determining factor. Our own experience would indicate that for buildings of ten, even twelve, floors, to carry ordinary live loads, concrete columns need be no larger than of structural steel fireproofed. Beyond that height we prefer to use the latter, though often enough we use structural steel for lower floors with concrete for the upper with results not only structurally practical, but economical as well. The Hide & Leather building is a sincere effort at producing an attractive building in reinforced-concrete. It would have been more successful in my opinion were its detail of better character and had some of the applied ornamentation been omitted. Its straightforward treatment of piers, its base and topping out are to be commended.

Considerable excellent concrete work has been done around Cincinnati and Dayton. The Beaver Power Company building in Dayton is an example. Entirely done in concrete, it is of interest in its simple variation of pier sizes, cornice and surface treatment. It is a very satisfactory structure architecturally, and so is the building for the American Tool Works Company in Cincinnati, by Stegner & Hughes, handled in a manner not unlike the Beaver Building.

Reinforced-concrete is at its best when used for low buildings as in Southern and Western places. The Presbyterian Hospital at San Juan indicates how beautifully the material lends itself to

UNIVERSITY CLUB BUILDING, LOS ANGELES, CALIFORNIA.
Allison & Allison, Architects.

such structures. This is concrete design at its best. Simple pier and wall surfaces, square and semi-circular openings, no cornice and plain belt course. Of similar character, though a high build-

ing, is the new University Club in Los Angeles by Allison & Allison. This, I believe, one of the best buildings we have in concrete. Its design is excellent in every detail and particularly noteworthy is the surface treatment which apparently exposes form marks without hesitancy. This building indicates the possibilities of reinforced-concrete for exterior use in civic architecture, an unlimited field in which it has been but little employed up to now.

An excellent example of good concrete design is the Harrison Garage, San Francisco, by McDonald & Applegarth. Apparently the surface is stuccoed, but there is beauty in its simplicity and well proportioned mass, which would not have been marred by the omission of the stucco.

The Bureau of Printing and Engraving at Washington is a monumental structure with its main facades of stone. It is questionable, however, whether they are better architecture than the inner courts which are of exposed reinforced-concrete. Certainly the latter serve the building better in a practical way. They are of exemplary design.

A building recently completed in Brooklyn for Kahn & Feldman by Buchman & Kahn has splendid character. It is the Industrial Building glorified.

As to residence work, it has been the general impression that houses built of reinforced-concrete are more costly than such of brick or tile. Many results, however, especially the work of Hueber Bros. of Syracuse, which is particularly notable, would prove the contrary. Furthermore, a number of systems of precast sections are coming into vogue which should make reinforced-concrete used in that manner very desirable.

Quite remarkable results have been achieved by Englishmen and a few examples thereof are exhibited in the booth.

Two very excellent English examples are the facades of the Tilling Stevens Motor Company at Maidstone and the Napier Motor Company at Acton. It is to be regretted that the surface is lined off to represent stones, the more so since the designs otherwise are so appropriate for concrete. The ornaments were precast and set into the forms before pouring of the concrete.

Since my own office has done a great deal of industrial work in reinforced-concrete, to which we have tried to give an architectural note, I trust that my referring to it will not be taken

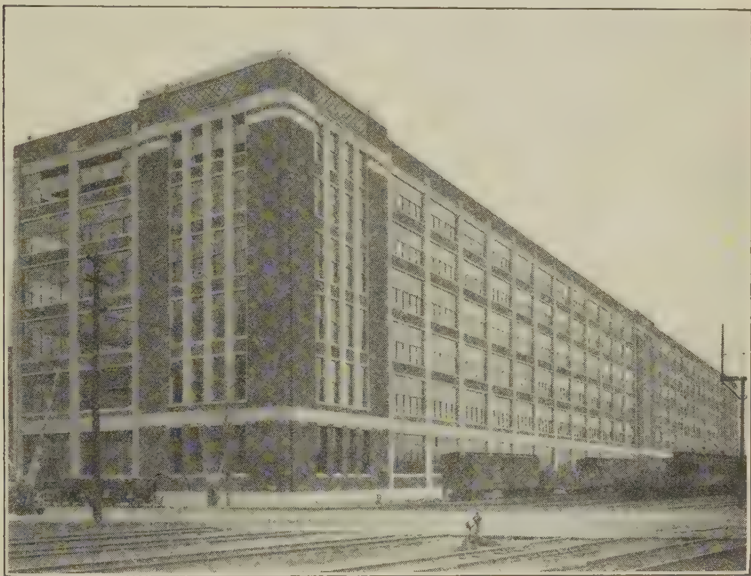
amiss. The automobile industry which to our good fortune established itself in Detroit some 20 years ago required large areas of manufacturing space with a maximum amount of light, rigid floors to carry fast moving machinery and buildings necessarily fireproof. Here was an excellent opportunity for reinforced-concrete. The Packard Motor Car Company was among the first to adopt the construction. Their main building faces our principal boulevard. In this an attempt at beautifying the exterior was made by concealing some of the horizontal structural members and framing the openings with brick work. The structure was only two stories high originally and two stories have since been added, which necessarily did harm to the design.

Our first problem which, because of its imposing dimensions, afforded a real opportunity, was the first unit of the Ford Motor Plant at Highland Park. The building is some 800 ft. long and 4 stories high. Here we used brick work as trimming, facing the piers of the entire lower floor therewith, also the corner and intermediate pylons. Between the latter we have above the lower floor the regular system of columns and lintels with a simple concrete cornice crowning the structure. In this building were used probably the first steel sash, now so generally employed, at least sash of standard factory type. They were shipped direct from England. We have felt and my colleagues have been kind enough to concur in this, that this building marked a step in advance in factory building, in that it attempted to make the building an architectural composition without sacrifice of practical essentials. Among other commissions entrusted to us are the plants of Hudson Motor Car Company, also those of the Continental and the Lozier Companies. In all of these buildings we have adhered to a combination of exposed concrete, some of it structural, with brick and cement colored tile inserts, with results that have won favorable comment. In the Hudson group, the Administration Building originally smaller than now, was executed entirely in white cement, the idea being to have its white mass emphasized by the darker brick trimmed factory building back of it.

In the Continental Motor Plant we indulged in more detail; to excess, we feel now, and were we to do it again we would omit much of it.

Our most recent structure of importance is the new Body Plant for the Studebaker Corporation in South Bend. Here we had a rare opportunity, the building being six stories high and over 1000 ft. long. We have in this increased the more usual brick pylons to brick pavilions with exposed concrete mullions.

In most of our work we have treated exterior concrete with Carborundum and given it several coats of cement paint, which in



BODY PLANT, STUDEBAKER CORPORATION OF AMERICA, SOUTH BEND, INDIANA.
Albert Kahn, Architect.

contrast to the brickwork proves effective. This matter of surface treatment is still a problem. How will it be solved? Left in its natural state untouched from the forms, cement washed, or rubbed with Carborundum, concrete at best is cold in color and uninteresting in texture. Stucco or other surface applications have proved on the whole a failure in most parts of the country, for they do not adhere because of difference in expansion and contraction and develop unsightly cracks. Attempts have been many to treat the surface by tooling or exposing the aggregate, but few have proved successful and none economical in cost. A ray of hope is afforded by

the remarkable work of Mr. Earley. His first efforts in connection with Meridian Park at Washington, D. C., gave promise, his latest, however, are a distinct achievement. By the use of aggregates carefully selected as to color and size and these exposed through wire brushing and washing with acid, he produces a surface and texture of surpassing beauty. Withal, it is no imitation of anything but true concrete, remarkable in its effectiveness. This is working along the right lines, and though Mr. Earley claims that a surface application by his process will positively adhere, designers will, no doubt, agree that such treatment at best is less desirable than a monolithic mass by itself, made attractive. Mr. Earley has done remarkable work and deserves the acclaim of all interested in the subject. We sincerely hope that he will continue his splendid efforts in advancing the art.

Now what of the future for reinforced-concrete? I am convinced that we have only made a fair start and that its development both structural and artistic will exceed any present expectations. Furthermore, I believe that concrete in its most direct form will grow in use for exterior facings. Gradually we shall accustom ourselves to form marks, we shall not only accept them, but take advantage thereof, we shall eliminate cement washing and rubbing even pointing and gain an artistic effect through the play of light and shade and emphasis of the monolithic structure which it really is. Furthermore we shall accustom ourselves to weather stains and make a virtue thereof, just as the grime of London only adds interest to its architecture. We shall pay more attention to design in mass, to interesting outlines and less to minor details. The way has been shown in a number of existing works. Others will carry on and continue to develop the new material, further proving that reinforced-concrete is the most notable contribution to the art of building since the structural steel frame came into existence.

I have not dwelt on the improvements possible in formwork or the many details of engineering. Nor have I mentioned all that is being done in precast concrete. These subjects are beyond the scope of this paper.

In closing, I can only express the hope that the good work of this society in diffusing knowledge and promoting the interchange

of ideas, may continue. There is much left for the Institute to accomplish. Concrete is by no means given its due today and much is possible with it that has not been undertaken. With general developments, improvements in cements, with better methods of constructing formwork, more liberal building codes permitting more economical though adequately safe design and construction—with a better appreciation of the inherent qualities of the material, greater and more general interest on the part of Architects the future of reinforced-concrete promises much indeed. May the Institute's good work continue and bear fruit in ever-increasing measure.

Chairman Humphrey.—*About the time this Institute was having its initial meetings which were described at this morning's session, it was my pleasure to meet in St. Louis a man who has been a very close adviser in all the various trials and tribulations which the speaker encountered as President during that time.*

The speaker has had the opportunity of following this man's own work and has viewed with admiration his progress as an expert, so it is with peculiar pleasure, particularly in view of the fact that he is the President-elect and will occupy this platform tomorrow morning, that the speaker presents Mr. A. E. Lindau, who will address you on

"*Concrete Bridges*"

Reinforced-Concrete Bridges.

By A. E. Lindau

President, American Concrete Institute.

THE new world opened to the architect and builder of buildings by the development of concrete and reinforced-concrete was likewise opened to the engineer and the builder of bridges. The vision of the bridge engineer as well as the architect was stirred by the vast possibilities of a building material which is commonplace today but was practically unknown a generation ago. For thousands of years we have built bridges of stone, brick, timber, and in the last century also of iron and steel. In the short span of a single generation, however, a new material has been developed and its use has been extended throughout the civilized world.

Bridge building is an ancient art. The Chinese are credited with building masonry arches 2000 years B. C. In fact, records seem to indicate that the Chinese used both slab and arch construction in their bridge building, and are entitled to rank with the ancient Assyrians as bridge builders.

In Persia over the River Diz we can see today an arch bridge of a series of 20-foot spans, 1200 to 1300 ft. long, still in a fair state of preservation and said to date about 350 B. C. The ancient Greeks built a bridge at Assos of stone slabs doweled together and supported by diamond shaped piers spaced about 10 ft. apart; the stone slabs were 20 in. thick and 24 in. wide. This structure is a

prototype of our present-day concrete slab bridges, especially of the "pre-cast" type.

To the old Romans, however, belongs the glory of having carried the art of bridge building to a greater degree of perfection than any of the older civilizations. In fact we owe to them not only examples of marvelous masonry construction and the development of the arch bridge to a degree scarcely excelled today, but also the discovery of the use of cement and the making of concrete. There are in fact grounds for suspicion that they realized the advantage of reinforcing their concrete structures. In the foundation of one of their temples bronze rods were found imbedded in the concrete, leading us to believe that in a rudimentary manner, at least, the Romans anticipated our modern reinforced-concrete construction by about 2000 years.

To complete our picture of the state of the art of bridge building handed down to us by these master-builders, let us stop for a moment and note the magnitude of some of their largest undertakings.

The bridges Marcia built in 145 B. C., 61 miles long with 12 miles of arch construction; Claudia, 50 A. D., 46 miles total length and 10 miles of this in arch construction; Anio Novis, 52 A. D., 58 miles total and 9 miles of arches; Alexandrina, 226 A. D., 15 miles total length, 7 miles of arches. In all, according to a summary by Professor M. A. Howe, a total of 63 miles of arch construction was built between 312 B. C. and 226 A. D.—a truly astonishing achievement and a testimonial of the degree of skill attained by these old engineers. It is not alone their skill in construction that challenges our admiration, but their ability to combine architectural beauty with their engineering proficiency that marks their work as extraordinary.

In the barest outline the foregoing represents our heritage from our early ancestors. The art of masonry construction as applied to bridges can almost be said to have reached its high water mark at the hands of the Romans, and for 1700 years no appreciable advance was made. As the light of civilization flickered and was almost extinguished during the Dark Ages, we can barely trace the story of concrete. Here and there an occasional example may be found, as in the case of a foot bridge at Amalfi on the Gulf of Salerno built by the Moors in the sixth century.

However, we can pick up the thread of the story again a century ago in the construction of a large concrete bridge over the Derdogne at Souillac, France, built in 1816, in which Roman or Pouzzolan cement was used. But the examples are few and far between until we reach the period in which Portland cement was discovered and manufactured.

This was an epoch making discovery in the field of bridge engineering as well as in architecture.

The next great step in the development of the art was the invention of reinforced-concrete. Whether a rediscovery of methods known to the Romans or not, the incorporation of steel bars or fabric in concrete opened a new era to the bridge engineer. Up to this time he was bound by the traditions and limitations of masonry construction. It is true that plain concrete offered enormous advantages in many directions over the older art of the stone mason; yet it was, after all, artificial stone and subject to the same limitations in design. The combination of the two materials, steel and concrete, produced a new material, marking a radical departure from old forms and methods. No longer was the designer bound by the conventional methods of arch construction. The strength imparted to the concrete by the reinforcement so materially decreased the weight of the structure itself as to make it possible to erect arch bridges of astonishing length of span and beauty of form. Not only that, but girder bridges and even trusses are added to list types available to the designer.

The credit of first designing a reinforced-concrete bridge may always remain a matter of controversy. Thaddeus Hyatt, an American, and Francois Coignet, in France, were early workers and experimenters in the field of reinforced-concrete and both understood the value of reinforcement. Joseph Monier, however, in a supplementary patent dated August, 1873, made specific claims for the use of his system of construction to building of arch bridges, and illustrates by diagram in his patent applications his ideas of reinforcing such structures. Monier's ideas seem a little crude and immature to us now, but they embrace the broad principles on which we have reared the art as we know it today. The original Monier system employed wire netting near the intrados (or lower face) of the arch only, but was later modified so as to use two systems of nettings—one near each face or surface of the arch ring. Many

Monier arches were built in Europe, one of the earliest of which we have a record being a foot bridge of 53 ft. span and 13 ft. width built in 1875.

Progress in the new method of construction, while a little halting to begin with, soon gained momentum, and we find that European engineers quickly gained confidence in reinforced-concrete and reveled in the freedom gained by it. So we find an arch bridge built in Switzerland in 1891 having three spans of 128 ft. in which the crown thickness is $6\frac{1}{2}$ inches. Again a bridge built in Germany in 1890 with a 132 ft. span and a crown thickness of 9.88 inches. As Mr. Thacher remarks in his paper before the International Engineering Congress in St. Louis in 1904 (just 20 years ago), "Such delicate dimensions would not be seriously entertained in the United States." While this remark clearly brings out the difference in European and American practice of twenty years ago the gap has been closed in recent years, and we can now show a multitude of examples as daring in design and "delicate in dimensions" as those found abroad.

Under the favorable conditions of economic advantages and freedom of design, reinforced-concrete arch design developed in various ways away from traditional forms of masonry construction. The first examples in Europe as well as in this country belong to the old Roman or Medieval type; with arch ring the full width of the bridge, parapet walls rising from the arch ring to the level of the roadway and back fill of earth to form the foundation of the road-bed. For moderate spans this type is still in common use and has some advantages, viz., the back fill absorbs the shock of moving loads and tends to distribute these loads to a fairly uniform intensity over the surface of the arch, the back fill also costs less than a concrete floor system above the arch ring. In long spans, however, the weight of the back fill imposes so heavy a burden of dead load that it becomes desirable to carry the roadway on superstructures of some kind. This led to the open spandrel type.

Open Spandrel Arches.—In this type the spandrel walls are omitted and the roadway is carried by a floor construction, which may be either of the beam and slab type or a series of small or supplementary arches. Here we find the designer taking advantage of some of the possibilities of reinforced-concrete, reducing the quantities of material and thereby the cost, and at the same time

producing a structure more pleasing to the eye. Although the structure in general appearance may resemble somewhat the older arch bridges of the double deck type, yet in detail it is a serious departure from previous forms. One of the earliest examples of this advance in design is represented by the three-span arch bridge over the Ybbs near Weidhofen built by G. A. Wayss & Co. in 1898, having a main span of 147 ft. and a crown thickness of 15½ in. Quite an airy looking structure, it must be admitted, and a remarkable one considering the date of construction.

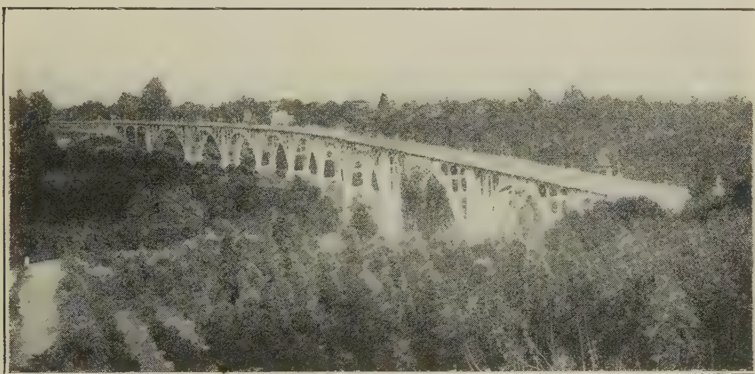
MONROE STREET BRIDGE, SPOKANE, WASHINGTON.

Designer, J. C. Ralston. Architect, Collaborating, K. K. Cutter. Span 281 ft., c t c. 300 ft. Rise 113.7 ft. Crown thickness 6.75 ft. Twin ribs. Deck to water surface 140 ft.

An interesting modification is illustrated by the highway bridge over the Colorado River at Austin, Texas. Here we find the open spandrel type bridge with cantilevered sidewalks which yield a further economy by reducing the length of the supporting piers. This is a distinct departure from the masonry and even plain concrete types. In neither of these older designs is it practicable to reduce the pier length by projecting the superstructure materially beyond the faces of the piers.

Ribbed Arches.—The genius of Hennebique in discovering the possibilities of reinforced-concrete is indicated by his designs of ribbed arches. For short spans the roadway slab is built in-

tegrally with the ribs, making the structure, in effect, a curved T-beam construction. In longer spans, however, dead-load of the structure can be saved by separating the roadway slab from the ribs by means of supporting columns. One of the most notable examples of reinforced-concrete bridges built in the early days is of this type. The bridge is located at Chatellerault, France, and built in 1899. At the time of its construction it was the longest reinforced-concrete arch span ever attempted, the middle span being 164 ft. and the two side spans each 131 ft., each span having



ARROYO-SECO VIADUCT, PASADENA, CALIFORNIA.

A reinforced-concrete arch viaduct 1468 ft. long and 40 ft. wide and 150 ft. high for highway traffic. Built for the city of Pasadena and the County of Los Angeles. Completed 1913. Cost $200,000. One 28 ft. roadway, two 5 ft. walks. Bitulithic pavement. One arch 223 ft., two 152 ft., six 113 ft., and small end spans. By Harrington Howard & Ash, Engrs.

an arch ring stiffened with four parallel ribs. The roadway of reinforced-concrete beam and slab construction is supported by columns which transmit this load to arch ring directly over the ribs. Here we still have the traditional arch ring, but the material in the ring has been distributed so as to stiffen or strengthen it along the lines of the applied load. The Chatellerault bridge stands out as a land mark in reinforced-concrete bridge design. The dimensions of all its members were refined to such an extent that we may say that we have a concrete structure that in appearance approaches a steel structure.

Isolated Ribbed Arches.—The Chattellerault type has been modified by omitting the arch ring between the ribs. In other

words, if it is of advantage to take a part of the material in the arch ring and make ribs of it, why not concentrate all of the arch ring material in the ribs? Under certain conditions of span and loading this advantage obtains, and as a result of this modification we have arrived at the "last word" in reinforced-concrete arch design. While examples of this most modern type may be found as early as 1906, it is only in the last few years that the design has

FIRST CONCRETE BRIDGE BUILT IN UNITED STATES, 1877.
Located in Prospect Park, Brooklyn. This photo taken two months ago.

become quite generally adopted, especially for long span monumental structures. In fact, we have in the United States today a number of notable bridges of this type; as, for example, the Arroyo Seco near Los Angeles; The Washington Memorial, built by Newcastle County, Delaware, as a memorial to the soldiers and sailors of Delaware who had taken part in the wars of the nation; the Detroit suspension viaduct in Cleveland; the Francis Scott Key over the Potomac; the Gilbert street bridge in Danville, Ill., and many others in connection with which data is not available.

Melan Arches.—A decided innovation in the matter of combining concrete and steel was introduced by Professor Joseph Melan of Austria-Hungary in 1892. The Melan arch consists of a concrete arch in which rolled steel beams or riveted lattice arch-ribs are imbedded. For short spans and rolled I-beams are employed and for the longer spans the latticed ribs.

The first Melan arch bridge built in the United States was erected at Rock Rapids, Iowa, in 1894. Following this, in 1897, the Kansas River bridge at Topeka of five spans with a total length of 693 ft. In rapid succession these structures were followed by others, so that by 1904—in a period of ten years—about 300 spans were constructed throughout the United States under the Melan, Thacher, and Von Emperger patents.

Hinged Arches.—One of the most startling innovations in reinforced-concrete design is the introduction of hinges for arch bridges. Dr. Von Emperger in a paper before the American Society of Civil Engineers in 1894 describes an arch bridge at Wurtemberg over the Danube having a span of 164 ft. and a rise 1/10 of the span provided with three cast iron hinges, one at the crown and one each at the abutments. The use of hinges seemed particularly applicable to Melan construction and many interesting examples of this type of bridge can be found in Europe. American engineers, however, have not adopted this device to any great extent, although some hinged arches have been built here.

Railway Bridges.—The railroad engineer approached concrete construction with due caution. The art was fairly well established before he felt justified in trusting the new material to the severe conditions he had to deal with. To the Illinois Central Railway apparently belongs the credit of being first in the field, as early as 1895 they built plain concrete arch culverts. We are also indebted to the Scotch for pioneer work. The West Highland Railway in 1898 built a number of plain concrete arches varying in span from 30 to 50 ft. One bridge at Borodale, however, had a span of 127 ft. A few years later American railway engineers became interested, so that by 1903 several railway systems had concrete bridge work under way. The D., L. & W. built a twelve-span bridge at Newark, and the Illinois Central boldly extended the field of concrete construction by building in 1903 the Bid Muddy arch bridge at Carbondale, Ill., three spans of 140 ft. long in the clear. The

Big Muddy design, considering time and conditions, was startlingly bold, but the traditional influence of masonry construction is in evidence in building the arch ring in sections or vaussoirs, in every respect like a stone arch, and making no attempt to use reinforcement. Similar in character but of somewhat lighter proportions is the Big Four bridge at Danville, Ill.

In total length of structure the Long Key viaduct of the Florida East Coast Railroad surpasses all modern bridges, and in this respect completes successfully with the old Roman viaducts. The bridge consists of a series of 50 ft. semi-circular arch spans having aggregate length of six miles. Of the notably long bridges may be mentioned also the Galveston Causeway. This structure provides for a double-track railroad line, double-track electric line, and a 19-ft. highway. It consists of a series of 70 ft. arches and has a total length of 2455 ft.

Girder and Slab Bridges.—The use of various forms of arch designs for bridges is after all largely a modification of earlier forms and consequently did not call for either the technical skill or the courage in breaking "new ground" demanded by slab and girder design. This venture awaited the development of the mechanics of reinforced-concrete beam action and the verification of the theory by tests. Hennebique reports that in 1893 he built his first girder bridge at Don, Department Du Nord, Paris. Apparently this phase of the art developed rapidly, for we find a Hennebique girder bridge at Yverdon with a span of 42 ft. built in 1896, at Lausanne in 1896 a span of 49 ft. and in 1903 a span of 66 ft. One of the boldest of this type is a girder bridge built at Milan in 1895 on the Walser Girard System, having a middle span of 84 ft. As an achievement in length of span it is surpassed by few structures today.* Exact data on the introduction of this type of bridge in the United States is not available, although examples of flat topped culverts for railroad work as early as 1899 can be found.

As knowledge and confidence in reinforced-concrete grew, bar reinforcement was substituted for I-beams, so that by 1904 highway bridges as well as railroad culverts of reinforced concrete slab and beam design were quite extensively used for moderate spans.

---

*It may be of interest to note that recently a reinforced-concrete girder bridge was built in Humboldt Co., Calif., having a span of 142 ft.

In fact by 1907 many railroads in the Middle West had adopted reinforced-concrete as standard construction for culverts and moderate slab spans. Of special interest is the track elevation work in Chicago by the Illinois Central and the C. M. & St. P. Ry., in which the road-bed is carried over street crossings on pre-cast concrete slabs.

The invention of the flat-slab or girderless floor for buildings has also been extended to bridges. A bridge in Colorado having a 38 ft. span is a notable example in the highway field, and in the railroad field we find the D. L. & W. has built a number of them where head room was at a premium.

Truss Construction.—Realizing that in long span girders the dead-load of the girder itself will establish the economic limit of span, open web or truss designs have been attempted and in many cases successfully executed. Particularly is this true of what is known as the bowstring girder. This is really an inverted arch in which the floor system is suspended by bangers from the inverted arch, and the thrust of the arch taken up by the bottom chord. The elimination of the thrust may at times make a very great saving in cost of supporting piers, as well as maintain the integrity of the structure in case of settlement of the foundations.

Summary.—The ground so far covered by no means exhausts the variety of forms developed up to present time, but enough has been given to indicate the wide range of designs available to the engineer, the wonderful flexibility of the material and the intense development that has taken place. Unfortunately data are not available to show by figures the increase in volume of concrete and reinforced-concrete bridge construction from time to time, but a hasty glance at the landmarks in the history of the art may serve to give us, in outline, a picture of the industry that has been built up. Keeping in mind that concrete was used by the Romans in their bridge work, that for seventeen centuries no advance was made, and that during this period only an occasional structure serves to connect the present with the past, we reach the early nineteenth century without any marked increase in activity. Then a few examples in the next fifty years following the invention of portland cement. By 1875 reinforced-concrete had been invented and the volume of construction experienced another sudden forward movement. At this time concrete had been introduced in the

United States by the building of a plain concrete arch in Prospect Park in Brooklyn in 1871. By 1889 the first *reinforced-concrete* bridge was built in the United States at Golden Gate Park, San Francisco. We can now measure our progress in years instead of centuries and a widening application of the art throughout the world. By 1893 it is estimated that 300 concrete bridges had been built in Europe, in the ten-year period from 1894 to 1904 in the United States about 100. During this same ten-year period Hennebique and his agents built over 400 bridges, and it is probable

WASHINGTON MEMORIAL BRIDGE, DELAWARE.
B. H. Davis, Engineer.

that at least that many more were built by all the other engineers and concrete specialists devoting themselves to this class of construction.

At the time of the founding of the American Concrete Institute twenty years ago, the industry was fairly well established in Europe and had gained firm footholds in this country. A number of types had been developed. Designers had boldly explored the unknown possibilities of concrete and reinforced-concrete. In the enthusiasm of youth they marched on where "angels feared to tread." Practice was years ahead of theory. In general, experience and designing instinct rather than sound scientific training guided the builder. In spite of the apparently large number of

successful examples that could be pointed to, the layman, and indeed the average engineer, was uninformed and hence skeptical of the "new-fangled notion." But the pioneer period was drawing to a close. A new period began in which a sound theory of design was developed and supported by a vast amount of research and experimental data. Economic advantage broke down the resistance on the part of the public to new methods and skepticism was forced to give way before an ever-increasing volume of facts.

While it may appear that the art was fairly well developed prior to 1904, this is due to the fact that we have attempted to review the development in all the years before that time. Every great industry is a process of growth, generally slow at first and subject to wide variation in rate of development. New ideas and methods take time to "sink in" and be accepted, especially by the public at large. Therefore, twenty years ago, despite the fact that scattered over the world one might find hundreds of concrete and reinforced-concrete bridges that most of the types that we know today had been tried out in a tentative way, nevertheless this was known only to a few who specialized in this work. As a matter of fact, the construction of a concrete bridge twenty years ago, even of moderate dimensions, was generally considered an achievement. The project was regarded as an innovation and grave fears and doubts had to be swept away before a favorable decision was reached. After the job was finished and the false work removed, those responsible for the design and erection would heave a sigh of relief and appear to be pleased that the expected had happened—the bridge was still there—and that their own doubts were groundless. Each new structure was a new experience and served as data for the design and building of the next one.

Indeed, experience was the guide rather than theory in determining the dimensions of the structure. The theory of the elastic arch had been stated by Weyrauch as early as 1879, had been developed and restated in various forms by later authors and was available in standard textbooks on mechanics. But the subject seemed to bristle with mathematical difficulties and the impression was common that it was quite as much an achievement to produce the design as it was to produce the structure. The bridge engineer had inherited the voussoir method of building arches from earlier builders reaching back to the Roman period, and the idea that a

stone or concrete arch was an elastic structure, subject to changes in form due to the applied load as well as expansion and contraction with changes in temperature, was not generally believed important.

In the past twenty years this situation has completely changed. The experimental stage has passed. We have in the last decade or two been stabilizing and consolidating theory and practice. We have learned more about the properties of concrete, its limitations as well as its possibilities, in twenty years than in all the time prior to that. On every hand we see reinforced-concrete bridges replacing older and less permanent structures. Memorial and monumental bridges of reinforced-concrete have become commonplace. Today reinforced concrete is a standard structural material. Methods of design are standardized and the common property of well-informed structural engineers, instead of the special, and, in some instances, the mysterious knowledge of a few specialists. Standard specifications and more or less standard methods of contructions are the rule rather than the exception. We are in the midst of a period of standardization. It is indeed the order of the day, and sharply marks the difference between our present situation and that of twenty years ago.

While we are apt to judge the future in the light of the past, it seems incredible that the next twenty years will see as intense and rapid a development as the last twenty. We shall in all probability develop some new types of concrete bridges, erect structures more imposing and larger in scale than those of today. We may reasonably expect improvement in the materials of construction, especially in the strength and reliability of the concrete. We may see the actual construction of such structures as the proposed Henry Hudson Memorial Arch with its 700 ft. span, more than doubling our present record of length of arch span. But beyond the boundaries of our present knowledge lies the land of tomorrow, and what we shall find there is a matter of speculation. Let us hope, however, that the next twenty years will show as large a measure of the achievement as the past twenty.

Chairman Humphrey.—*The next subject appears on the program with the statement that,—*

> *"The property which permitted concrete to lift its head like a tree also permitted it to sent roots down into the earth."*

Perhaps none has been more successful in developing the art of sending the roots down into the earth than the gentleman who is to address you next. He is an active member of this Institute and has been accustomed to seeing concrete structures grow; as a matter of fact, he is a man of achievement. We are fortunate in having Mr. Max M. Upson to speak on

> *"Concrete for Foundations and for Water Front Work."*

Concrete for Foundations and Water-Front Work.

By Maxwell M. Upson

Vice-President and General Manager, Raymond Concrete Pile Co., New York City.

Concrete has like attributes for all its uses. The same characteristics that permit it to rise in towering superstructures enable it to find support and anchorage in penetrating deep into the mysteries of the earth. Each forward step of engineering seems to run parallel to, or converge toward, collateral developments. These are so related that were they not united into one broad highway, the road of advancement would be lost. Cheap fuel gives cheap steel and cheap cement. These two products, especially combined, permit high and heavy structures—much heavier than the supporting power of many grounds. A new, permanent and cheap foundation is needed. Behold the advent of the concrete pile and the concrete caisson!

They, like many of their concrete brothers, are indeed the children of cheap cement. Old Judge Economics passes with unerring wisdom on every off-spring of the human brain. He condemns with heartless cruelty all those who cannot pay their way, and gives unbridled rein to those who meet our economic needs.

And it is here pertinent to observe that the long period of adversity through which the cement industry passed was not an altogether unmitigated hardship. It has bred a lusty and large family of children who are now industriously supporting their fertile parent. The concrete pile and concrete caisson are two of these children and they promise to grow as fast and as favorably as the wisdom and fairness of progenitor will permit.

The concrete caisson, like the concrete pile, comes from the substitution of concrete for wood in a long-known engineering principle. The theory of the diving bell formed the basis of the development of the first air caisson. As early as 1820 Lord Cochrane devised and patented a system of air caisson contruction which brought into play most of the factors now generally used. His development grew out of the problems confronting the engineers in the building of the Thames tunnel. Later in 1839, a Frenchman by the name of Triger encountered quicksand in the sinking of a coal mine shaft in Belgium. The water and sand baffled all attempts to carry on the construction. As a result, work was halted and experiments were carried on in conjunction with another engineer in Paris to determine whether it was possible for a man to work under air pressure.

Triger and his co-worker subjected themselves to 20 lb. of air in a receptacle which exploded during the test. However, sufficient information was obtained so that Triger was able to design a lock and working chamber which resulted in efficiently finishing the mine shaft. So far as we know, this was the first air caisson constructed.

We are indebted very largely to General William Sooysmith for the adoption and perfection of the air method of foundation construction in this country. He began his operations with a bridge at Savannah, Ga. The development of the war brought General Sooysmith north to Chicago, where he carried on consulting engineering work involving many important air caisson foundation constructions.

Until the beginning of the last decade, most of the caissons were built of timber. Cast iron and steel have been used from time to time and even at this date the former is almost exclusively employed in under-river tunnel construction.

It is of moment to us who are now celebrating our twentieth anniversary to know that it was not until 1906 that the first all-reinforced-concrete caisson foundations were installed. These constituted the support of the new municipal building of the City of New York. The methods, design and apparatus were developed by The Foundation Company under the patents and supervision of Daniel E. Moran and J. W. Doty. The suitability of this mystic

BOOTH AT TWENTIETH ANNIVERSARY MEETING DEVOTED TO CONCRETE FOR FOUNDATIONS AND WATER FRONT WORK.

combination of cement and aggregate to this particular phase of foundation construction is now so manifest that we, from our present engineering viewpoint, can little understand why it was so slow of adoption.

And, parallel with this comes the slowly but surely moving sentiment of engineers toward the reinforced-concrete crib as against the old wooden crib. The shaking off of old prejudices and the opening up of the light of progress represented by economy and better constructions, is rapidly driving the foundation engineer to an almost complete substitution of the crib made of con-

crete for the crib constructed of timber. This is admirably illustrated in the foundations of the bridge which is soon to unite the cities of Philadelphia and Camden. Here single cribs are being sunk having cross-sections 140 x 40 ft. and extending down 110 ft. below ground level. These cribs have twelve excavating openings and as their edges penetrate through the hardpan near their ultimate foundation seat, they form a complete cofferdam, cutting off all incoming water and making dry excavations and the placing of dry concrete possible.

So, too, has come the acceptance of the concrete linings for shafts, tunnels, and all types of excavation which require permanence, rigidity and strength.

The concrete pile came into being before the relationship of concrete and steel was thoroughly appreciated by the engineer. In 1897, Alfred A. Raymond, a bridge builder working on the railroads in Nebraska, patented what is known as the "Raymond" system of piling. He aimed to produce a pile with a length of life equivalent to that of the superstructures which he was building for the railroads in that Middle West country. This method proved simple and effective. It consisted of driving a collapsible mandrel, encased in a sheet steel shell, collapsing and withdrawing the mandrel after proper penetration is attained, and filling the steel-lined hole with the concrete. So far as the speaker has knowledge on this subject, Mr. Raymond was the first man to develop and produce a practical and economical method of placing concrete piling. He is undoubtedly the father of the well-known system of cast-in-place concrete piling.

In 1903 Richard J. Beale developed another method of driving cast-in-place piles which consists essentially of driving a steel pipe, either plugged with a follower or with a cap, filling the pipe with concrete and withdrawing the pipe. This is the essential principle involved in the placing of the well-known Simplex and MacArthur piles.

From this beginning came two classes of piling—the "Cast-in-Place," as represented by the Raymond, Simplex & MacArthur, and the "Precast" or "Reinforced-Concrete" pile, which is based on the magic relationship between steel and concrete. On these two branches hang all the fruit of the industry, and rare and interesting have some of the specimens been."

Before the advent of this new foundation element, engineers having to do with piling thought in terms having distinct limitations; the top of the piles must be carried down to an elevation assuring permanent saturation; the loading of each pile must be limited because of the size of the natural growth of timber. The concrete pile, in removing these limitations, has opened up new avenues of real economic significance. Before its advent, the capacity and size of certain structures were, of necessity, limited, because there was not sufficient space to accommodate the necessary number of supporting wood piles. Certain ground was considered valueless for the reason that there was no known method of providing an economical foundation. The soil was too soft for spread footings, the permanent water level was not available to protect wood piling, and hard sub-soil lay at too great a depth to use caissons. It is natural that the concrete pile should step forward to meet these economic requirements.

This concrete unit may be increased in size and in length to meet all practical needs. It is independent of water line. It may act as an element to carry its load entirely by friction, or it may penetrate to a substantial underlying stratum. It may be of any cross-section which its designer desires. Its function as a column is, therefore, subject to the whim of the engineer. This is well evidenced in the support of piers where deep water causes each pile to serve as a column. Under such conditions, the cross-section dimension of the normal growth of timber frequently limits the safe loading of wood piles to from 4 to 5 tons each, while large precast piles in the same depth of water may be designed to carry as much as 100 tons each. Here opens up a new possibility to the harbor engineer since it permits the building of heavy storehouses on piers which, before the advent of the concrete pile, of necessity must have been placed on the adjacent shore.

Such a design is admirably illustrated in a pier and warehouse recently built at Halifax, N. S. Here the water is deep and the range of tide is extreme; yet it was possible to place a heavy, reinforced-concrete, 2-story warehouse on the pier, the main floor carrying a live load of 1000 lb. and the upper floor 500 lb. to the square foot. The building of the warehouse and pier would have been structurally impossible through the use of wooden pile. On the wood pile, 10 tons would have been the maximum safe load, while

on the 24 x 24 in. reinforced-concrete piles, the working load approximated 100 tons each.

It is obvious that such developments have opened up a new field to the terminal and harbor designer. He not only has old and recognized structural limitations removed, but is able to accept these new principles on the basis of an economic saving. Sometimes these savings are not so apparent when structure against structure is the only basis of comparison, but when all collateral costs are considered, the economic advantage looms large.

The so-called collateral costs have to do with the use of land heretofore considered impractical of development because of foundation expense, the saving in the site area due to the placing of the warehouse on the piers, the reduced cost of handling, less maintenance, insurance and depreciation. In addition to all these advantages comes a frequently overlooked item of economic significance—the saving of time in construction. This saving sometimes brings large dividends to the owner. The analysis of all these figures, when properly collected, is frequently the determining factor in the design. No structural engineer can afford to disregard them. He must be a master, not only of the actual costs of the work, but also of maintenance, of insurance, of depreciation and of the often forgotten construction, time factor.

Recently the speaker was called before a harbor commission to present the advantage in the use of concrete piles in open pier construction as against the contemplated creosoted wooden pile. It was found, on the analysis of the annual costs, including maintenance, depreciation and insurance, that the concrete pile structure saved in these three items \$45,000 per year over the wood structure. The actual difference in cost was only \$50,000. In other words, the increased initial investment gave a return to the city of almost 100 per cent annually. And in addition all loss of occupancy and use risk had been removed. It is needless to record here the action of the Commission.

The real significance of the time-saving element is perhaps worthy of consideration. Recently, a large manufacturer, about to erect a branch factory on ground already partially occupied, discovered that larger and heavier buildings required special foundation provisions. The concrete pile met his requirements. To use a cast-in-place pile as against a precast reinforced type meant a

saving of 60 days in time. Since the potential earnings of this plant were rated at $100,000 per year, it is obvious that he saved approximately $16,000 in the choice of the quicker type of construction, which, incidentally, was more than the total cost of the piles for the foundation.

And to all the above must be added the saving in dollars and cents which comes in every operation where these new foundation elements are used. It is not unusual to encounter conditions where the total cost of the concrete pile foundation is 50 per cent of the cost that would have been involved in the old wooden pile and deep footing method.

So we see some of the fruits borne by these magic concrete roots which penetrate the mysteries of the underground. They may be named Reduced Initial Cost, Increased Structural Possibilities, and Saving in Construction Time. They constitute the essential virtues of this construction octopus.

There is another essential to which we, as engineers, are wont to turn our critical eye, and that is sufficiency. Before this assemblage, a brief on the merits of concrete seems hardly necessary A material rot-proof, fireproof, unaffected by water or dryness, and structurally efficient, ought to be accepted as a substitute for those which have many or all of these weaknesses. And it is pleasing indeed for me to record that it is. Yet it has taken twenty years of persistent, expensive and heartbreaking education on the part of the producers to secure for the concrete pile, recognition as a standard unit of construction. Some of the responsibility for this conservatism on the part of the engineer in accepting this new child into the family lies at the door of the producers. Improper methods and careless workmanship have given notable examples of difficulties and failures. Yet the percentage of such troubles has been so low and the reasons therefore so obvious that the thinking engineer has used them not to condemn but to improve the industry.

And this brings us to the consideration of a subject of very pertinent interest to all members of the American Concrete Institute, namely, the durability of concrete when subjected to sea water. The very notable failures of concrete under such exposure have naturally caused the engineering profession to proceed cautiously in accepting it in harbor and pier construction. Many

notorioús examples of such failures have occurred in structures dotting our coast line. It has been the speaker's privilege to examine most of these, and in almost every instance where deterioration is found, it is apparent that one or more of the fundamental laws which we now know underlie the producing of a permanent concrete has been broken. Cement, with its compounds, when first produced, was heralded as a savior from the wanton destruction of the teredo and limnoria.. Little or nothing was known by either the engineer or the contractor of scientific mix, impermeability, density, waterproofing, proper reinforcement, proper curing and the score of other fundamentals on which the production of a permanent concrete is based. Is it to be wondered that there are failures? They sailed forth onto an uncharted sea, without compass or sextant; yet, strange as it may seem, some of these structures bear silent testimony to the possible durability of concrete in such service if properly designed and built.

This combination of cement aggregate and steel in the construction of bulkheads and piers has opened up virgin fields of design. Never before, in approaching the solution of these problems, have we been able to consider monolithic elements which act as a unit and convey a multiplicity of stresses in sections of almost limitless size. The piling or sheeting to which these slabs and beams are connected may be so rigidly joined to the superstructure that all elements unite to resist the pressures to which the structure, as a whole, is subjected. This principle makes it possible to revise the old and established rules of design and break forth into a new field. Greater strength and rigidity are secured and economy of construction is thereby attained. These principles, scientifically applied, have demonstrated in many instances that an all-concrete bulkhead or pier can be constructed at an equal or lesser cost than the long used creosoted wood structures.

Here is an opportunity that may well stir the ambitions of the engineer who has imagination. Try to count in your mind the value of the pier substructures in the harbors of this country. The aggregate sum is worthy of attention even in these days when billions are discussed lightly; and yet our insurance companies tell us that the risk in fire on these piers amounts to from 2 to 3½ per cent of their value. Our records of maintenance on wooden structures show that from 2 to 4 per cent is a low cost. These

two items, without taking into account deterioration or replacement, aggregate a yearly cost to this country running into many millions of dollars.

As sponsors for this one child of the great concrete industry, may not our hearts swell with pride that we may have a part in bringing about at least a partial saving of this gigantic waste?

And to this alone we may not give all our pride. The business of concrete piling today, young as it is, aggregates over 3,000,000 lin. ft. per year, which, converted into wood piling, equals over 7,000,000 lin. ft. This means that the concrete industry today is saving a lineal footage of wooden piles which, if placed end to end, would extend approximately from New York to Kansas City. In other words, reducing the saving to terms of average timber land, we find that more than 4000 acres of virgin timber is saved each year by this industry. At this period in our civilization when our lumber supply is so rapidly disappearing, it is a satisfying thought to know that we producers of concrete are having an ever-increasing part in this conservation.

Perhaps it is not unbecoming or immodest for us on this anniversary day to dream of what the world would have been without these foundation elements of concrete. When we contemplate the sky line of our modern city and consider how essential these foundation methods are in making possible the construction of our mammoth buildings, we bow in humble homage to the accomplishments and service of this mysterious synthetic stone. It veritably supports the framework on which much of our social relationship hangs. Without high buildings our present large cities would have been impossible. The efficiency of business relationship and manufacturing accomplishments would have been placed on a scale immeasurably lower. Even the relationship between the home and the place of business would have been altered to a degree beyond our present conception. Concrete has had and always will have its sociological as well as its economic influence.

And now for the future of this industry. It lies in two hands.

First, the producer. No industry grows unless quality is made the paramount aim of those who guide its destiny. The profits today, this year, or for 5 years, have little significance if the product will not stand the test of time. Emerson has said: "If a man write a better book, preach a better sermon or make a better mouse-

trap than his neighbor, though he build his house in the woods, the world will make a beaten path to his door." So it is with our industry as with all others.

Not only must the producers bend their energies to scale the heights of perfection in the product, but they must have the co-operation of the engineers and the manufacturers of cement. Without good materials no workman can do his best and the producer of the concrete which goes beneath the ground must, of necessity, rely on the manufacturer to give him the proper cement with which to work. The constructor should apprise the cement manufacturer of the service to which his material is to be put. And he, in turn, must know enough of this product to supply that which is suitable to the service. Their interests are inviolably mutual and any betrayal of faith on the part of either party may shatter these wondrous structures which now promise to rise to great heights. We stand together, engineers, constructors and manufacturers. Where a misstep is made, it is duty and privilege to stop in the busy pursuit of a living and lend a hand and advice so that we may each feel that we have a part in building up a thing that is so worthy of our ambition and pride.

Chairman Humphrey.—*In 1905 about the time the Structural Materials Laboratories of the U. S. Geological Survey were getting under way in St. Louis the speaker was introduced here, in Chicago, to two young men who were students in Lewis Institute and who at the time were working on a thesis on The Effect of Steam Curing on Cement Blocks. The progress of one of these young men has been characterized by persistent effort to perfect what he undertook as a student, and it is with peculiar pride this afternoon, because this young man began to receive his first insight into concrete in the St. Louis Laboratories which were under the direction of your presiding officer, that he presents to you Mr. R. F. Havlik who will speak on*

"Concrete Products."

Concrete Products.

By Robert F. Havlik

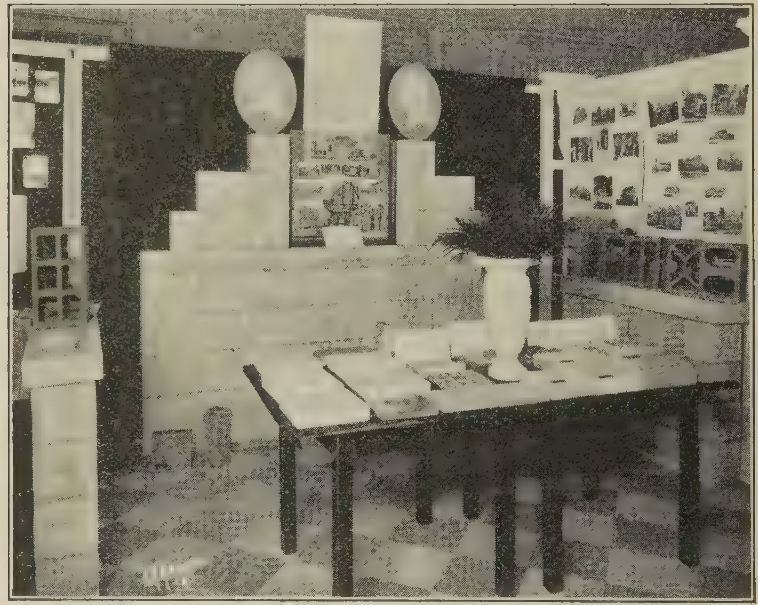
Mooseheart, Ill.

The limited time given each speaker on this Anniversary Day program prohibits going into great detail on any of the subjects assigned. My comments on concrete products will, therefore, be of a very general nature. I believe the points which would prove of greatest interest are statistics of the scope, growth, etc., of the concrete products industry.

Probably the first form of concrete product ever made was the concrete building block. Precast concrete blocks were made as far back as the middle of the nineteenth century, but did not come into common use until the beginning of the present century when many ingenious men conceived the idea of making hollow building blocks. While some were inventing machines for making hollow concrete building blocks, other enterprising men devised ways and means of manufacturing other concrete products, such as drain tile, sewer pipe, floor tile, and roofing tile. Machines for making such products were first placed on the market between the years 1900 and 1907. The development since the early part of this century has consisted of converting hand machinery into automatic machinery so as to reduce labor cost to a minimum. The principal rea-

son why concrete building blocks were not popular prior to this century was that the early blocks were all made solid. The invention of machinery for making hollow building blocks put the industry on a competing basis with other building materials.

The most common form of hollow building block is the one with three cross webs and two air spaces. A one-time popular form was one with two rows of staggered air spaces. Other popular

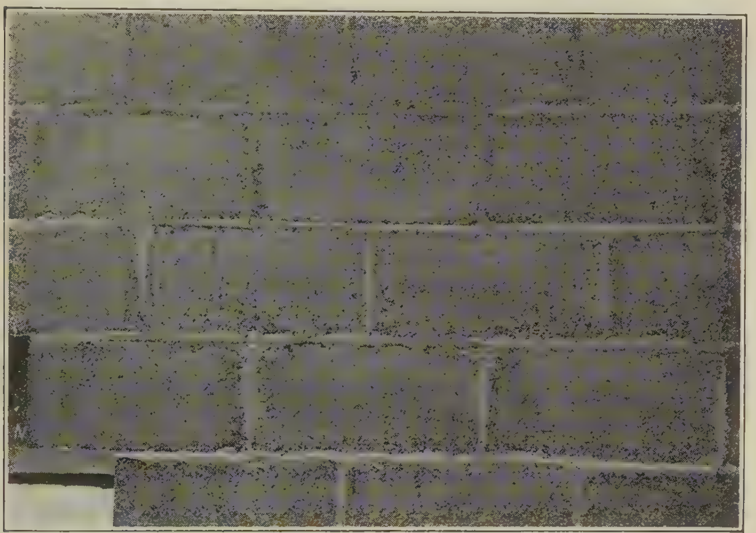


BOOTH AT TWENTIETH ANNIVERSARY MEETING DEVOTED TO CONCRETE PRODUCTS.

forms of blocks are the so-called "two-piece" blocks. Some of these are tied together with metal ties, the ties being inserted in the blocks while they are manufactured. Others are laid up with connecting metal ties, and still another form of block is the interlocking "T" shaped two-piece block. All these, excepting the staggered air space blocks, have been very popular to the present day; the staggered air space block is seldom seen now.

Many and interesting have been the arguments in favor of each particular type of blocks. It would be out of place for me to

express any preference at this time on such matters. Suffice it to say that good high grade walls can be built with each of the many types of blocks now on the market, providing the contractors using the products, and the manufacturers making them, use every possible effort to put out a high grade job. In the first place, good concrete blocks cannot be made unless the manufacturer is honest enough to put in the requisite amount of cement. In addition to this, he must have a sufficient knowledge of the concrete industry



A CONCRETE BLOCK WALL READY FOR STUCCO APPLICATION.

that will enable him to use a sufficient amount of water in his concrete. Finally, if he will put the necessary expense into manufacturing the product, he will produce units with beautiful textures that will satisfy the most exacting.

Most building blocks made in the past have been made in rock-faced designs, in imitation of pitched stone. This custom retarded the industry for many years, and has done more harm to it than any other one factor. In recent years, however, there has been a decided tendency to make plain blocks with granite facings. Rock-faced blocks have been used principally for foundations under frame buildings, and will doubtless continue in favor for this pur-

pose for a long time to come, but for high grade construction plain blocks with granite textures are usually used. Trim stone made of the same textures has become very popular, so the granite-faced blocks are bound to benefit greatly thereby, as the two can be used to excellent advantage in combination.

Hollow concrete building blocks are usually made in 16 and 24 in. lengths, and for walls 6, 8, 10 and 12 in. thick. Thicker

A TYPICAL CONCRETE MASONRY HOUSE.

walls are produced by using various combinations of these blocks. The most common size of building block is the 8 x 8 x 16 in. block, which can usually be found in the foundations of two-story residences, and in small cities for the walls of two-story residences, including basement walls. In most localities concrete block walls for foundations and inexpensive buildings are cheaper than brick walls, as can readily be seen from the chart on display in one of the exhibits. This, of course, holds true only of the common concrete block. High grade granite faced blocks are more expensive than

common brick, but will compete with the corresponding good grade of face clay brick backed up with common brick.

A small conception of the tremendous quantity of concrete building blocks being used at this time can be had from the following statistics, for which I am indebted to the Portland Cement Association:

In the year 1921 the equivalent of one hundred and seventy-five million 8 x 8 x 16 in. concrete blocks were made in the con-

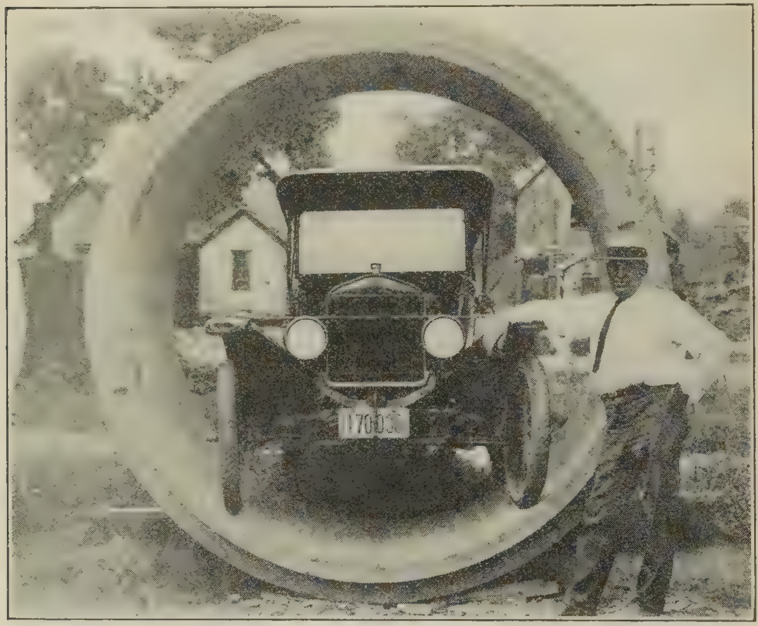
A TYPICAL CONCRETE STAVE SILO BUILT BY THE WATERLOO CONCRETE CORP., WATERLOO, IOWA.

Approximately 200 plants are producing concrete staves with an annual output close to 10,000 silos. Concrete staves are also used in the construction of corn cribs, grain tanks, and coal pockets.

crete products plants that have made annual reports to the Association. Considering the fact that a large percentage of manufacturers failed to send in these reports, it is fair to assume that the actual number of blocks turned out by all the concrete products manufacturers greatly exceeded this figure. This number of units was increased to three hundred million in 1922, and three hundred and fifty million in the year 1923. The 1923 production required the enormous quantity of seven million barrels of cement. It is of interest to know that most of these blocks were produced in the Central and Eastern States. With the coming years, the industry is bound to spread throughout the South and West, so that the consumption will double, or perhaps even treble, in a short time. This

will be brought about more rapidly by constant advertising of high grade building blocks made with facings in which the aggregate has been exposed by scrubbing or acid treatment or some similar method.

A big field that has been overlooked by most manufacturers is that of ashlar veneer in competition with face brick or cut stone.



SECTION OF CONCRETE PIPE, 96-IN. INTERNAL DIAMETER, USED IN THE CONSTRUCTION OF A SEWER IN OKLAHOMA CITY, OKLA.

Pipe were made by the Independent Concrete Pipe Company of Tulsa, Okla.

High grade white cement granite veneer building blocks 4 in. thick, or less, can be sold at a big profit at one-third to one-half the price of cut stone, and the profits to the manufacturer will be many fold that made on ordinary concrete building blocks. Architects are eager and willing to use such units. I find that this sort of unit finds favor with most architects, and if worked in combination with concrete granite trim stone it will prove a source of big profit to the manufacturers, and at the same time such units will effect a big saving to the owners of the buildings in which they are used.

There is no doubt but that white face concrete granite building units, whether they be in block size or trim stone size, are superior to any other type of masonry material, excepting natural granite. They are far superior to Bedford stone, in that they do not soil readily and can be cleaned with muriatic acid and water at a slight cost, whereas most natural stone can only be cleansed satisfactorily with a sand blast at considerable expense.

Concrete granite trim stone is extremely popular at this time. I can well remember how difficult it was ten or fifteen years

AN EXHIBIT OF CONCRETE FLOOR TILE.

ago to even obtain an audience with the average architect in behalf of concrete trim stone. Today it is a common practice to ask for alternate bids on concrete trim stone in competition with natural stone and terra cotta.

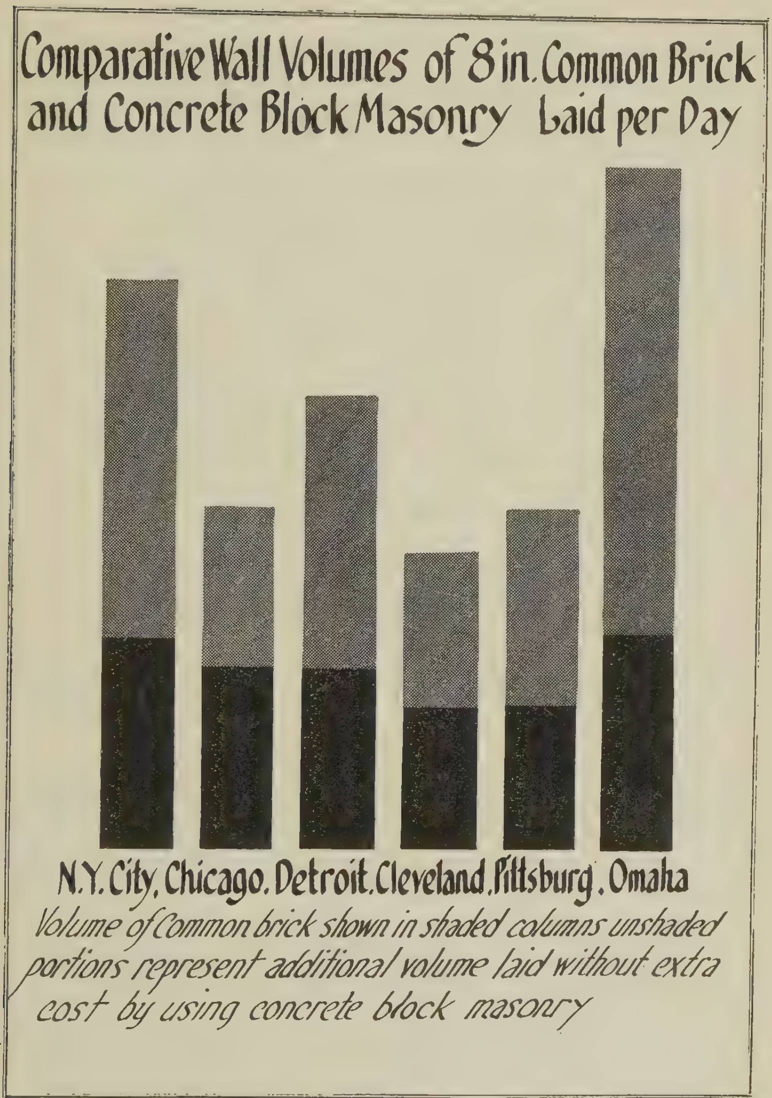
In recent years another building unit, known as the "light weight" building tile, has been developed, and this is meeting with considerable favor. No less than ten million of these units were made in the year 1922, and over twenty million were made in the year 1923. While this is a small figure as compared with concrete building blocks, it must be remembered that these units have only been on the market for a few years, whereas the hollow building blocks have been on the market for a quarter century. In view of

this, it can readily be seen that light-weight concrete building tile are destined to play an important part in the building industry of this country.

Concrete brick have had a varied history. It has been a very difficult matter to produce concrete brick at a profit to the manufacturer in competition with common clay brick. In sections of the country where clay brick are scarce, concrete brick can be produced profitably, but in sections where clay brick are more common it is difficult to make concrete brick profitably excepting with high grade automatic machinery. In the matter of face brick, however, the situation is entirely different; such brick can be sold profitably in competition with a moderately priced face clay brick. In some districts white cement granite brick are becoming quite popular. The reports from such concrete brick plants as made a report for the year 1923 to the Portland Cement Association show a production for one year of over one hundred and fifty million brick. The cement required for these amounts to three hundred thousand barrels. The concrete building tile produced in 1923 consumed two hundred thousand barrels.

Concrete roofing tile have not made as big strides as concrete building blocks, tile and brick, but during the past few years there has been a decided improvement in the situation. There are over fifty high grade producers of concrete roofing tile in this country, producing over eighty thousand squares of roofing tile annually. valued at about $1,000,000. Most of the concrete roofing tile made years ago was not as strong as clay tile, and this retarded the industry, but that which is now being produced is a superior quality that closely resembles the clay product, both in appearance and strength, and sells at about 60 per cent of the price of clay products.

Concrete drain tile, sewer and water pipe are becoming more popular from year to year. In fact, the increased popularity of these products is almost beyond comprehension. While concrete pipe has been generally used in this and foreign countries for over eighty years, the greatest progress has been made in this century, and most of it probably in the last decade. Concrete drain tile and sewer pipe are more uniform in quality than clay products, and, on the average, stronger, and usually they are also cheaper, even



COMPARATIVE WALL VOLUMES OF 8-IN. COMMON BRICK AND CONCRETE BLOCK MASONRY LAID PER DAY.

in the clay districts. Sewer pipe are manufactured in two classes—plain and reinforced. The plain pipe is produced in standard sizes from 4 to 24 in. and the reinforced pipe in sizes from 24 to 108 in. internal diameter. One company alone has sold over five hundred miles of reinforced-concrete pipe for sewer construction. Reinforced-concrete pipe are also used for culverts in sizes from 12 to 14 in. Such use has become a standard practice by the principal railroads, and nearly all of the State Highway Departments. In 1923 such pipe was placed on an equal basis with cast iron pipe as to permanent strength and economy by a committee of the American Bridge and Building Association. Reinforced pipe is also being used extensively for irrigation work and water supply systems. The State of California at Delhi alone has installed an irrigation system of over two hundred miles of pipe from 12 to 36 in. internal diameter. Concrete pressure pipe for water supply system is being used quite extensively. One of the largest installations on record is at Tulsa, Oklahoma, consisting of fifty-two and a half miles of 54 and 60 in. pipe. Concrete drain tile have been used principally in the middle West, in standard sizes ranging from 4 to 72 in. internal diameter. One of the largest jobs on record was awarded in 1923 in Minnesota, consisting of four hundred and thirty-six miles of tile 25 to 34 in. internal diameter. Seventy per cent of this mileage was concrete embracing principally the larger diameters.

In these few moments, I have endeavored to touch upon the most interesting and important features of the concrete products industry. The markets of the country have just been touched, and the use of these products can be increased tremendously by concentrated effort. As a whole, I do not believe the concrete products industry has used as high grade salesmanship in advancing its interests as have other competing products. There is a bright future ahead for the industry as a whole in all its various branches if as good a brand of salesmanship is used in selling these to the public as our competitors use. High grade concrete trim stone and building blocks of granite texture, or equal, have a most promising future because of the fact that there is no more pleasing masonry construction than the ashlar type. Such masonry is conceded as the best for most purposes. In the past, only a small portion of such masonry has been built up of concrete units.

Comparison of Cost of Laying Concrete Block and Clay Brick Masonry					
	Common brick laid per eight hour day.	8"x8"x16" Concrete block laid per eight hour day	Common brick required to equal wall volume of Concrete block	Ratio of wall volume of Concrete block to common brick laid per eight hour day.	Relative labor cost to lay 8" Concrete block wall compared to equal volume of brick wall.
N.Y. City	1500	250	3250	2.17	46%
Chicago	1200	150	1950	1.62	61.7%
Detroit	1000	200	2600	2.6	38.5%
Cleveland	800	130	1690	2.05	48.7%
Pittsburgh	800	150	1950	2.44	41%
Omaha	1200	300	3900	3.25	31%

COMPARISON OF COST OF LAYING CONCRETE BLOCK AND CLAY BRICK MASONRY

A survey of 77 representative cities throughout the United States shows that the cost of laying a given wall area with concrete block is only 42 per cent of the cost of laying the same wall area with brick.

As a parting message, I, therefore, urge all concrete stone and block manufacturers to compete most vigorously for this type of masonry construction, which, after all, is the most profitable to the manufacturer. In these closing statements I am emphasizing the concrete blocks and trim stone, as I feel they have not come into as common use as have the various forms of concrete pipe. The average engineer is ready and willing to use concrete pipe on an equal or preferred basis with other pipe, and a vigorous effort on behalf of building blocks and trim stone will bring their use to the same happy state.

CHAIRMAN HUMPHREY.—*The speaker has always felt that the next person who will address us is a protègé of the American Concrete Institute, and that its members do not really appreciate how much of a pioneer he is in this very important field of the artistic treatment of concrete, although it was before this Institute that his work was first exploited. The work that he has done and is doing is illustrated in the exhibits in the other Hall and it augurs well for the future; the speaker believes that in that future, because of its great merit, this work will be referred to as Earley Concrete.*

It is a great pleasure to present Mr. John J. Earley, who will speak to us on

"Architectural Concrete."

ARCHITECTURAL CONCRETE.

BY JOHN J. EARLEY

ARCHITECTURAL SCULPTOR, WASHINGTON, D. C.

ARCHITECTURAL concrete is a concrete which meets the requirements of the architect and which is approved and used by him as a building material. The architect has absolute authority and is the sole arbiter on questions of appearance in the building industry. Only those things which received his approval find a permanent place in it. To force a way past him is useless. There is nothing to be gained but the shell of ostracism or the bitter cup of disappointment. It is just and right that such absolute authority should rest in the architect, because only in him is the full knowledge of the industry. To him belongs the treasure which experience has lain up through the ages.

It is hard for some to understand why the architect should have power to reject any material because of a poor appearance and they wonder how he describes as poor an appearance which is thoroughly satisfying to another. But one man's judgment is not as good as another's. The architect's opinion is founded on the common experiences of many men through many generations. The knowledge which he possesses may not be in scientific form, but it is true knowledge nevertheless. Scientifically expressed knowledge may aid in speculative thought and in the rapid development

of technique for new materials, but it is not absolutely necessary for either.

Knowledge like truth is simple. It is complicated only in its expression. The knowledge possessed by a fine craftsman differs in no way from that of a great scientist excepting in form. When Michael Angelo built the dome of St. Peter's Church in Rome he knew the stresses in that structure and took care of them efficiently

BOOTH AT TWENTIETH ANNIVERSARY MEETING DEVOTED TO ARCHITECTURAL CONCRETE.

and well. But I doubt whether he could have made the calculations in such form as to meet with the approval of modern engineers. When the vaulted roofs of medieval cathedrals were propped by flying buttresses, the designers with intuitive knowledge, with the knowledge of a craftsman correctly translated the stresses. I am sure they could not have calculated them, nor have expressed them on a diagram, and I wonder how many engineers there are today who can do so. During the world war, a striking example of what I mean was brought to our attention: The United States decided

to build a number of concrete ships, and inquiry was made of the world's best authorities on ship building for scientific knowledge of the stresses set up in ships in high seas, and none was found. Do

SHRINE OF THE SACRED HEART, WASHINGTON, D. C.

you think that the race of men through all its history has gone down to the sea in ships built by those who knew nothing of them? Such a question is ridiculous. They did know! The ship builder

SHRINE OF THE SACRED HEART, WASHINGTON, D. C.

and the marine architect had a craftsman's knowledge. The experience of ages had left to them real and practical knowledge. The recent work of certain scientists has clearly shown the normal eye to be a correct instrument which gives the sensations proper to the stimuli received. Therefore, we may conclude that the architect, when he rejects a material because it has a poor appearance, is neither blind nor mad, but is exercising a prerogative which is justly his, and which he is qualified to use.

SHRINE OF THE SACRED HEART, WASHINGTON, D. C.

Within the past twenty years concrete came to the architect and asked for a place in the building industry. It said, "I am young and strong, will work faithfully and for low wages." The architect replied, "All this I know, but your appearance is below my standards, therefore, I will not employ you." Concrete decided to work in the building industry in spite of him, but some how things did not go well and concrete was to recognize the authority of the architect and the need for a better appearance. Only a long story could tell of the efforts which have been made and the de-

vices which have been employed to improve its appearance. These efforts have been made along traditional and I must admit logical lines. The first movement is always to present a new material in the guise of an old. I do not mean you to understand that this is not proper. It is.

When electricity came into general use for illumination the mechanics of its installation were in no way standardized and the nature of its light was little understood. The fixtures which had been employed for gas and candle lighting and which through many years had been developed into artistically acceptable forms were of necessity used. Only recently, after the mechanics of installation have been standardized and the nature of the light has been better understood, have electric light fixtures assumed distinctive form.

The first development of an artistically acceptable appearance for concrete was the work of those who presented concrete in the form of stone. They are to be praised and encouraged. They made the necessary preliminary effort and will hold what has been gained while a true technique is being developed. Within the past few years concrete stone has been made into a fine product and has been able to meet artistic requirements in a manner equal to the natural stone which it simulates. In this form it has served a useful purpose and will enjoy popular favor until an appearance peculiar to concrete has been developed and put into general use. In all probability the time required for this will be sufficiently long to insure a just recompense to those who have been pioneers. We cannot, however, accept concrete stone as the full measure of concrete's ability to assume a good appearance any more than we can accept an electric candelabra as the ideal electric light fixture.

When you realize that considerable knowledge of the material and reasonably uniform methods for handling it must precede any useful work on its surface treatment or appearance and when you recall how short a period of time has elapsed since concrete has been treated as a plastic of great mobility and not a granular solid to be pushed and tamped into place, you will understand what I mean by saying that the development of a good appearance for concrete has been rapid. Within recent years, certainly within the past five, there has been an earnest effort to apply all available information to the problem. It has attracted men who can bring to it not only the trained sense of the artist and architect, but also

the knowledge of the craftsman and scientist. Through the co-operation of the members of this Institute we have learned much of the nature of the materials in concrete and of their control.

The elements of a good appearance are three, namely, good form, color and texture. And the experience of the past few years has taught that concrete of any form, color and texture can be made by the proper selection of the materials, particularly of the

SHRINE OF THE SACRED HEART, WASHINGTON, D. C.

aggregates, and by the effective control of them while in a plastic state. With regard only to its appearance, let me suggest that concrete be thought of as an aggregate, which is held in place by the least possible amount of hardened cement paste, and which before the hardening of the cement was flowed into place in a vehicle of water. This gives the idea that it is the aggregate which takes the form and gives the color and texture, that the cement is a binary material and has no part in the appearance, and that the water is a carrier which places the material with the least amount

of work. This idea is the most valuable fruit from the past twenty years of work on concrete surface treatments and it is our stake for the adventures of the future.

Although concrete is a plastic material which hardens in a mold and although the forming of concrete is a casting process, nevertheless; concrete has certain peculiarities which prevent the technique developed for other plastic materials from being applied to it. When the dry materials of concrete are mixed with water they will in time set or harden, but they do so in a different way from other casting materials. Plaster of paris, for instance, when mixed with water and cast in a mold, will not segregate or lose volume, but on the contrary will increase a little in volume and by this ensure complete filling of the mold and a sharp cast. In addition it will set quickly. Concrete does not act in this way. When placed in a form or when cast in a mold there is no increase of volume, there is no stability of mass. The solids tend to separate from the water, to settle towards the bottom of the mold, also to fall away from the vertical surfaces and the underside of the horizontal ones. Water immediately fills the place left by the moving aggregates and the result is a dull unattractive casting. Castings made of concrete usually contain a greater volume and weight of material than comparable castings in any other material and require a longer time to set or to fix the volume of the material against changes. All of these things when not controlled by a proper technique naturally cause the form of concrete to fall short of architectural requirements. These shortcomings have unfortunately been very general, indeed, that architects have come to accept them as necessary and have allowed modifications of form, have omitted architectural mouldings and reliefs because fitting execution could not be expected in concrete. Contractors have encouraged them in this belief and have by this means avoided technical difficulties. Some of the experiences of my studio would be humorous if they were not sad. Architects submit their problems of concrete surface treatment with apologies, with fear that that their drawings are too elaborate even though simplified to the point of crudeness, and with hope that we will do our best to retain what little is left of the esthetic.

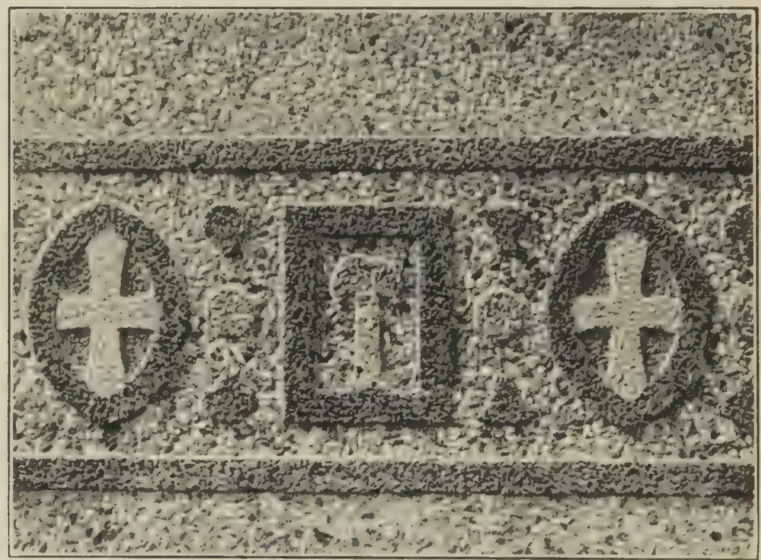
The color and texture of concrete are in a similar state of disorder and from a similar cause, namely, misapplied technique.

While color and texture are only adjuncts of form nevertheless they are to the artistic sense necessary adjuncts, just as the sensation of color, although not necessary to vision is the fullness of sight. Of all the materials and methods of coloring paint is the commonest and the technique of painting is probably the best known of the decorative crafts. This probably accounts for the efforts many have made to apply it to concrete. With pigments coloring is usually done by adding to a base, such as white, material of another hue. The base is thus changed in hue and chroma, according to the kind and amount of material added. Experience has taught that a base should be neutral, that is, it may range in value between white and black, but it may not have an appreciable chroma of any hue, particularly of a hue opposed to the colorant.

The coloring of cement by methods similar to those employed for pigments has not been successful because cement, even white cement, is not a neutral base. It has color sufficient to interfere with that of an added pigment. The difficulty is aggravated by the chemical activity of the cement which attacks the structure of many of our most desirable pigments. Furthermore the addition of finely ground pigment weakens the cement, therefore; the addition of an amount of pigment exceeding a small per cent of the volume of cement is not permitted, and as a result concrete cannot be given a hue of great strength, a hue approaching in strength that required for adequate decoration.

I do not mean to be discouraging by the difficulties which I have pointed out. They are of the past rather than of the future and I have reviewed them for the sole purpose of impressing you with the desirability of a technique proper to concrete. The fundamentals of such a technique have been developed within the past few years. Much must be added before the control of concrete will constitute a craft, but I think in time this will come about. As state before, the chief fruit of our labors is in the recognition of the aggregate as the major factor in the appearance of concrete. It is by constructing a skeleton of aggregate the volume changes, settlements and segregation are prevented, and the necessary perfections of form are secured. It is by causing the aggregate to occupy a very great part of the surface that predetermined color and texture are obtained. Coloring is not so difficult by means of aggregate as by means of pigment. Aggregates are not impaired

by the action of cement, the strength of chroma is not limited, and the character of the concrete will be permanent. A great wealth of color knowledge, all that of the impressionist or pointillist school can be immediately applied to concrete if we consider each grain of aggregate as a spot of color placed in just a position to other spots, all of which will blend in the air to a hue of even value and chroma. As a result of the experiences of the past twenty years I do not think the American Concrete Institute would be hasty if



SHRINE OF THE SACRED HEART, WASHINGTON, D. C.

it announced that the problem of giving a proper appearance to concrete has been solved and that subject to further development the technique of concrete surface treatment has been indicated.

Within the next ten years I expect to see concrete a fully accredited artistic and architectural medium. Because of my peculiar position as an architectural sculptor I am able to understand the position of the architect and to respect his arbitrary stand. I also have a craftsman's knowledge and am able to see the possibilities of concrete in art and architecture, its great facility, its exquisite form and color, and its limitless textures. This forecast for the

future is not, therefore, an enthusiastic hope, but a reasonable expectation.

A new building material, one of major importance, after it has gone through its novitiate and has been accepted usually begins to exert an influence, to give its own character to the work in which it is used. That concrete will be no different in this respect from other materials is proved by those artists and architects who are using surface treated concreted to a considerable extent. Certainly we may expect concrete buildings both in their structure and appearance to develop an individuality equally as marked from that of standard masonry as the latter is from frame.

I think there is as great a desire on the part of the architects that concrete be speedily given a good appearance as there is on the part of the cement and concrete industries. Every architect realizes that the cost of building is out of balance with economics and that for expediency many of the properties of his art must be sacrificed. He knows that these sacrifices have been made to an extent which threatens decadence. Such periods of artistic suppression have occurred before and have been relieved only by an economic change, by something which made it again permissible to properly decorate. Armies returning from the crusades brought memories of graceful minarets and beautiful domes, they also brought hands a plenty and the influence of both is easily seen on the architecture of Europe. When we think of the saving in the labor required to form and place a plastic mass over that required to form and place a solid mass and of the wonderful possibilities which better knowledge will give to concrete may we not conclude that this is the economic change which will allow a fuller and greater development of structural and decorative forms than did the return of those men from the Crusades.

CHAIRMAN HUMPHREY.—*When the history of this period is written, an outstanding figure among those who will have left their impress in the field of engineering, one whose research and investigation of concrete and reinforced-concrete have contributed so much to the industry that it is known the world over, one beloved by all who know him, a Past-President of the American Society of Civil Engineers, the American Society for Testing Materials and many others, than whom no one can speak with greater authority, will be Professor A. N. Talbot, whom it is my honor and very great pleasure to present to you and who will speak to us on "Engineering Research."*

ENGINEERING RESEARCH IN CONCRETE AND REINFORCED-CONCRETE.

BY ARTHUR N. TALBOT

PROFESSOR OF MUNICIPAL AND SANITARY ENGINEERING AND IN CHARGE OF THEORETICAL AND APPLIED MECHANICS, UNIVERSITY OF ILLINOIS.

AT THIS Twentieth Anniversary Meeting of the American Concrete Institute, much has been said of the development of concrete construction during the lifetime of the Institute. While concrete had been in use for many years before, and notable structures of various kinds had been built, the extent of its use in the nineteenth century was relatively small, as is evidenced by the output of American Portland cement, 5,500,000 bbl. in 1899, as compared with 137,000,000 bbl. in 1923. Foundations and the interior of massive parts had constituted its larger field of usefulness. With the opening of the twentieth century, however, reinforced-concrete, or concrete-steel or armored concrete, as it was commonly called, began to be of interest to the engineers and architects of the United States. It had made a start in Europe before that and a number of engineers there had learned much of the technique of this form of construction. Even in this country a few like Ransome in California had done pioneer work, and Hyatt, an American, had done early experimental work of some value. Many Americans had made use of it in small ways. But except in arch bridges the real start in the United States was made about the beginning of this century, a start that was destined to be followed by a most wonderful development in variety and nature of structure and of usefulness, and by a most marvelous expansion in the volume of reinforced-concrete construction, as well as in the use and usefulness of concrete generally.

In judging of the part that engineering research has played in this expansion and this development, it will be well first to consider the state of the art, the diffusion of the knowledge, and the general attitude toward concrete and reinforced-concrete that existed twenty or twenty-five years ago. Looking back twenty-five years it is not out of place to say that there was a general lack of technical knowledge of the principles of reinforced-concrete construction among engineers, architects and contractors in this country, as well as a dearth of experience in the practical side of the construction, and that there were many mistaken notions prevalent. In the earlier days I have known of reinforcement being placed on the wrong side of the member and of most absurd attempts at reinforcing and ludicrous errors in design. I hope I shall not be taken to mean that there were not some who were skilled and skillful in design and construction. The profession as a whole, however, was not informed on this new art, and some had mistaken notions and others special prejudices against this form of construction. Besides, a new art must prove itself sound, must do things to gain the confidence of constructor and owner and insurer and banker. It must demonstrate its correctness with certainty, its utility with satisfaction, and its safety and economy with conviction. And a new art does not burst full-blown into action; it must grow and develop and be trained towards perfection. I trust then that before I am through it will be apparent to all that engineering research in the technical side of reinforced-concrete has had a most important part in the development and extension of reinforced-concrete construction in the last twenty years, and that research will be thought worthy of your commendation on the occasion of the twentieth anniversary meeting of the American Concrete Institute.

I wish there were time and opportunity adequately to record the accomplishments of research in the field of concrete and reinforced-concrete in this period. With the limitation put on me, that is impracticable; I can hope only to give some impressions of the work and its meaning.

An agency which greatly aided and stimulated research in reinforced-concrete in the early part of this period was the Joint Committee on Concrete and Reinforced-Concrete. This committee, which held its first meeting in June, 1904, was formed by joint

action of the American Society of Civil Engineers, the American Society for Testing Materials, the American Railway Engineering Association, and the Association of Portland Cement Manufacturers, and a few years later representation was given also to the American Concrete Institute. A report of the subcommittee on Plan and Scope made at the time of the Engineering Congress in St. Louis in August, 1904, recognized clearly that it was vitally necessary to have extensive and adequate tests made to cover the field of concrete and reinforced-concrete on which to base conclusions and recommendations for design and construction and for regulating practice in this promising type of construction. A subcommittee on Tests was formed, whose most active members were Messrs. Hatt, Humphrey, Thompson, Turneaure, and the speaker. For some years this subcommittee outlined and suggested and stimulated research. Through their exertions the hearty co-operation of college laboratories and other laboratories throughout the country was obtained. The Structural Materials Laboratory of the United States Geological Survey at St. Louis, the laboratories of the University of Illinois, University of Wisconsin, Purdue University, Ohio State University, Massachusetts Institute of Technology, and various others made valuable contributions to knowledge, and later still other laboratories added contributions in many ways. The information so obtained, together with that derived from other sources, including foreign research, helped to enable the Joint Committee on Concrete and Reinforced-Concrete understandingly and intelligently to make the progress reports of 1909 and 1912 and the final report of 1916—documents that, while they were attacked as being both radically unsafe and unsound, and unduly and inordinately conservative, time has shown on the whole to be sound and constructive and to have exercised most important and beneficial influence on engineering practice in this country through these years. This committee, of course, was only one of many influences that stimulated engineering research.

I wish there were opportunity to relate in a historical way the many principles that have been established and the many points of practice that have been accepted, all based upon the research work of this period. Time will permit only a reference to the type of questions that required consideration. In beams at-

tention had to be given to the permissible tensile working stresses in reinforcing bars for different kinds of structure and different forms and qualities of bars, and to whether the tensile strength of the concrete may be utilized for the resisting moment of the beam. What working stress in compression may be allowed in the concrete, or better, what percentage of reinforcement for the given working stress in tension is permissible? How shall web stresses be calculated and what allowance shall be made for the various forms of web reinforcement—a most difficult and vexing question, particularly before measurements of the strains developed in the web reinforcement had been made. How much is the available bond resistance of a bar, and in what way does it depend upon form and size? What values of positive and negative bending moment shall be specified under the varied conditions existing in buildings and other structures? How shall floors be calculated, and how do their loads distribute themselves to and along the beams and girders? How do columns act and what is their strength? What is the effect of spiral reinforcement, both on the ultimate strength and on the useful strength of the column, and what of the combination of longitudinal and spiral reinforcement? Must columns resist bending put into them by the floor system and how may provision be made? How do footings act and how may they be designed? Has the flat slab a mysterious and wonderful strength which baffles analysis? What specifications may best be made for the strength of concrete itself? In details of construction, how close may bars be spaced both among themselves and to the sides of the member, and what provision shall be made for forms and bracing and false work? All these and very many other questions needed answering. Many of them now seem trifling, others trite, still others are yet far from being fully settled. All needed tests, information, real experimental facts. It is true there were some, as strangely there are still, who persisted in basing their views solely on their own analytical inner consciousness.

The photographs and drawings in the exhibit of Engineering Research are representative of investigations in concrete and reinforced-concrete made in this period of twenty years, more fully of the first part of this time, researches that altogether contributed greatly to the progress of this new art. At the risk of seeming to

overlook many important contributions, it may be of interest briefly to refer to some of the problems undertaken.

In this country before 1904 a little experimental work had been undertaken, notably by Lanza, Hatt and a few others, which had been very helpful in various ways, but perhaps most largely as a reconnaissance into the method of testing reinforced-concrete beams. The tests of large beams made in January, 1904, at Rose Polytechnic Institute by Howe, ably assisted by Condron, and those made at the University of Illinois in the spring of 1904 gave what was probably the first reliable experimental evidence brought out in the United States on the position of the neutral axis in reinforced-concrete beams, and with the careful and complete measurements of strains and full description of the action of the beams under load gave confidence in the methods of testing and the applicability of the results to practical work. The tests at the University of Wisconsin in 1904 gave clear evidence that the tensile strength of concrete gives way at low loads and that this tension is not an element in determining the strength of beams. Of other tests in great variety made in various laboratories in the years following, reference may be made to tests on limiting compressive stresses, methods of web reinforcement, tee beams, and reinforcement with plain and deformed bars. The Structural Materials Laboratory of the United States Geological Survey at St. Louis under the direction of Humphrey early produced a large amount of valuable and timely data. The tests of the University of Wisconsin under Withey were valuable contributions. Of the many beam tests made at the University of Illinois those of Larson on continuous beams having a great variety of forms of web reinforcement (yet unpublished) are worthy of mention.

But research is always found to have incidental by-products. Did not the tests of 33-ton slab beams made for the Illinois Central Railroad in Chicago and viewed by a hundred engineers give to these engineers and many others an appreciation and a confidence in this new form of construction otherwise not easily gained? And did not the sight of the comparative tests of large cast iron culvert pipe and reinforced-concrete pipe give these same engineers a new idea of the applicability of this material? Similarly, the early tests of buildings—the strains in the concrete and the steel being measured first by Lord, followed by Hatt and Slater and Gonnerman and Richart and others—these tests not only gave needed in-

formation on the action of the structural members of the building, but conveyed to designer and constructor and owner a confidence in reinforced-concrete that was lasting, even though, as many will remember, it was difficult to overcome the prejudice against those fine cracks that the whitewashed surfaces exposed to the view of the layman, cracks which the research man knew from the beginning must accompany the loading of such structures.

To consider other technical lines: The work of Abrams on bond will long remain a classic. The tests on wall footings and pier footings, also made at the University of Illinois, have given the whole profession information and fundamental conception of the action and resistance of footings that seem to have proved very useful. The investigation of columns at the University of Wisconsin, University of Illinois, and Lehigh University, and that important series of the American Concrete Institute made by its Committee at the Pittsburgh Laboratory of the Bureau of Standards and the tests of McMillan on shrinkage changes added needed knowledge of this important member. I wish I could talk in detail of the column tests. The tests of reinforced-concrete frames by Abe at the University of Illinois and the later tests by the Emergency Fleet Corporation, described in the Proceedings of the Institute by Slater, threw much light on forms of construction that will be used more and more in the future.

In the important field of flat slab construction, the many tests of buildings made in various places and the tests on experimental slabs at Purdue University and elsewhere, as well as the unpublished tests of cantilever slabs at the University of Illinois, have given noteworthy information, and these and the more recent analytical investigations of Slater and Westergaard have changed the status of the flat slab from a mystery or a miracle to a substantial reality.

Only passing reference may be made to a variety of research work of other kinds—the fire tests of the Underwriters Laboratories and the Bureau of Standards, in which Ingberg and Hull had an important part, the valuable tests of stucco construction by Pearson at the Bureau of Standards, and the recent extensive tests of the United States Bureau of Public Roads under Goldbeck and the various State Highway Departments, as well as many minor tests made at various college and other laboratories, all of them aiding in the progress of the art of construction. Nor can time

be taken for a statement of the work done in recent years on that most important subject, the elements influencing the properties and quality of concrete itself, a topic that still deserves the earnest attention of many laboratories, notwithstanding the extensive and important work done by Abrams and the Structural Materials Research Laboratory and by various other laboratories.

Attention is called to another feature of the exhibit, a collection of bulletins published by universities and government laboratories, recording the results of research in concrete and reinforced-concrete. (University of Illinois 23 bulletins; University of Wisconsin 7 bulletins, and sample publications from other schools and the Bureau of Standards.) The public spirit of these and other state universities and the general government in providing for this research and disseminating the information to the public in this way is deserving of commendation. Such publication and the proceedings of the American Concrete Institute and other societies and the engineering periodicals are replete with evidence that engineering research has contributed greatly to progress in reinforced-concrete in the past two decades.

I have said little of the work of the last few years. Research has become so widely extended and so prominent in engineering activity that all are familiar with it. Notwithstanding this there must be regret that recognition can not be given here to the very many valuable contributions to knowledge made in recent years.

May it not, then, be concluded confidently that engineering research has been a large factor in the development of reinforced-concrete in this twenty-year period? And the end is not yet. Engineering will continue to need and use ingenuity and experience and judgment, and rules and precedent and vision, but more and more as time goes on will it be dependent upon engineering science, which in turn depends upon research. Herein lie future opportunities for the American Concrete Institute. Cement needs study—we are still ignorant about it in spite of boasts. Concrete needs study—its ingredients, the methods, the requirements to insure durability. Reinforced-concrete needs study—it is far from perfect. By continuing to encourage the spirit of research in all these directions and by assuming responsibility in determining and advocating right methods and condemning improper practices, the Institute will be adding greatly to its service to constructional interests and the public.

CHAIRMAN HUMPHREY.—*The next topic is "Making Good Concrete." That of course is the rock upon which the Institute will survive or perish. One of the men who has been outstanding in the devotion of his life to the study of the art of making better concrete, a man who is well known to us all and who hardly needs to be mentioned by me, Prof. Duff A. Abrams, who will tell us something of what he knows on the subject.*

MAKING GOOD CONCRETE.

BY DUFF A. ABRAMS

IN CHARGE STRUCTURAL MATERIALS RESEARCH LABORATORY,
LEWIS INSTITUTE, CHICAGO.

THE type and adaptation of structural materials is one of the best criteria of the development of the race. Reinforced-concrete is the newest arrival in the long list of materials which have been used by man for shelter, protection, convenience, or ornament.

Reinforced-concrete was used in a small way both in Europe and in the United States during the last quarter of the nineteenth century, but it has seen its greatest utilization in the United States during the past twenty years.

Early Development.—It is difficult to review the progress of the past twenty years in concrete making without referring to earlier developments.

Concrete may be defined as an intimate mixture of hydraulic cement, aggregate, and water, which is permitted to set and harden under favorable conditions. Without qualifying adjectives concrete is now understood to refer to portland cement concrete.

This term was applied as early as 1780 to a mixture of lime and boulders or crushed rock; later it was used to refer to mixtures made with natural cement.

The use of mixtures of pebbles or stone chips with some binding material is as old as civilization and dates back to the prehistoric use of mud for this purpose.

Modern use of hydraulic cement dates from the work of Smeaton in the construction of the Eddystone Lighthouse in 1756-9; the next great step was the invention of portland cement by Joseph Aspdin in England in 1824. Progress in the utilization of hydraulic cement was very slow; several seawalls, breakwaters, etc., were constructed by English and French engineers during the '40s and '50s. The first great impetus to the use of portland cement came in 1856, when Grant adopted this material in the mortar for the system of sewers then under construction in London.

The first uses of concrete on a large scale were in the construction for the armies and navies of England, France and the United States—dry docks, seawalls, jetties, etc. The next extensive use was in canal construction—retaining walls, locks, etc.

General G. A. Gillmore seems to have been the first American engineer to make a careful study of concrete. These studies were made immediately following the Civil War.

The exhibit which accompanies this paper shows some of the significant features of the literature of concrete making. These books and pamphlets may be considered as milesposts on the road to good concrete.

Adaptability of Concrete.—Concrete owes its principal advantages as a structural material to its high strength in compression, ease of placing, wide distribution of constituent materials, low cost, satisfactory resistance to fire and other destructive agencies, and to its good appearance.

Concrete differs from other structural materials in that it is actually manufactured at the site of the work, usually with unskilled labor and with little expert supervision. Like all manufactured materials, the final result depends on carrying out each step in an intelligent manner. Due to the almost endless variety in the structural characteristics and in the size and grading of aggregates, it is obvious that concrete is in many respects a very complex material, although in its essential principles it is quite simple.

Steps in the Making of Good Concrete.—The following steps must be considered in making good concrete:

(1) Selection of materials.
(2) Design of concrete mixtures.
(3) Mixing of concrete.
(4) Transporting.
(5) Placing.
(6) Curing.

BOOTH AT TWENTIETH ANNIVERSARY MEETING DEVOTED TO MAKING GOOD CONCRETE.

The requirements which must be met by cement are covered by the Standard Specifications and Tests of Portland Cement of the American Society for Testing Materials. It is interesting to note that the requirements set forth in these Specifications reflect the results of the experiences of the entire world. The specifications for portland cement of practically all civilized countries are essentially the same as those now standard in the United States. There has been a marked elevation in the minimum requirements for cement during the past twenty years, especially in fineness and

briquet strength. Cement tests are more generally made than formerly.

There has been a marked advance in requirements for aggregates, although our ideas have changed in some important respects with reference to the value of tests for wear, hardness, etc., which were originally developed for rock for macadam roads.

Earlier practice in concrete making was based on the use of arbitrary mixtures such as 1 part cement, 2 parts sand, and 4 parts crushed rock or gravel. Scientific studies have furnished the basis for design of concrete mixtures in such a way that the materials at hand can best be adapted to the requirements of the work.

The studies of effects of size and grading of aggregate on the strength and other properties of concrete were begun by Feret in France in the early '90s; similar tests on concrete mixtures were published in this country in 1906. During recent years many studies have been carried out which have further extended our knowledge of the scientific principles underlying this phase of concrete making.

There has been a marked tendency toward the general use of aggregates of larger size—particularly in concrete road work; it is not uncommon to see aggregates up to 3 or 4 in. used for this purpose.

Tests and experience have shown that sea-water, bog-waters, waters containing sewage, etc., may be used for mixing concrete with good results.

Twenty Years of Concrete Research.—The practical use of concrete antedated our knowledge of its real properties. The passing of time has not always marked a forward movement in the art of concrete making. The widespread use of reinforced-concrete and lack of real knowledge of the function of the materials which go to make up concrete, led, in many instances, to the use of too much mixing water, and hence, as we now know, to inferior concrete.

The scientific studies of concrete and concrete materials in the United States are practically coincident with the life of the American Concrete Institute.

Only scattered tests of concrete had been made prior to 1900; practically all such tests were made at the Watertown Arsenal. Earlier investigators gave a great deal of attention to reinforced-concrete research; the history of development in this field has been

sketched by Professor Talbot. The scientific study of concrete was a later step.

The investigations of Fuller and Thompson, Sabin, Wig, Edwards, Hatt, Goldbeck, Pearson, Withey, Talbot, and many others have produced results of far-reaching importance. Many of the reports of these researches form a part of the Proceedings of the Institute.

The somewhat tardy development of a proper technique of concrete testing was largely responsible for many false steps in progress toward better concrete. There is a close relation between the output of testing machines and the production of good concrete.

Probably the most significant recent advance in concreting practice has come about as a result of the general recognition of the fact that the quantity of mixing water (as compared with the quantity of cement in the batch) is the controlling factor in determining the quality of concrete.

Machine mixing of concrete is almost universally done on important work, although good results can be secured by hand-mixing if proper attention is given to details.

Care must be exercised in the transporting concrete from the mixer to the forms in order to prevent segregation. The widespread use of chutes for distributing concrete on the work is one of the notable features of the past twenty years' development. Chuting is an economical method of transporting concrete, but unfortunately, it places a premium on the use of an excess of mixing water, which generally produces a concrete of low strength and inferior resistance to weather and other destructive agencies.

Qualities of Good Concrete.—The most desirable qualities of good concrete are:

(a) Strength.
(b) Resistance to wear.
(c) Resistance to weather.
(d) Resistance to fire.
(e) Resistance to sea water and other destructive agencies.
(f) Permeability.

The strength required of concrete is dictated largely by the type of structure under consideration; it is practicable to design and secure concrete having any designated strength up to any 4000 lb. per sq. in. at 28 days.

The resistance of concrete to wear became an important factor upon the advent of the concrete road. The destructive effects of weather on all structural materials is too frequently overlooked in discussions of the quality of concrete; we are beginning to realize that weather, particularly in latitudes above 40° is one of the most destructive agencies that concrete must withstand.

Greater care is being taken today than ever before to see that concrete is protected against drying out until the cement has hardened and the concrete has attained a considerable proportion of its ultimate strength. Proper curing is especially important in work exposed to high temperature and winds or in thin sections exposed to summer climate.

Quality of Concrete as Influenced by Type of Structure.—Mass concrete was the earliest form of concrete construction. Here it was feasible to take advantage of the known principles of design of concrete mixtures to a greater extent than in other types of work. There is no necessary restriction on the size of aggregate which may be used. It is also feasible to satisfactorily place drier mixtures than is the case in reinforced-concrete construction.

In reinforced-concrete construction the workability of the concrete, the size and grading of the aggregate, etc., must be dictated by the difficulties encountered in the placing. This generally requires a smaller size of aggregate and a wetter mixture than would be used for mass concrete.

A widespread use of concrete in road construction has exerted an important influence in raising the general quality of concrete throughout the country. For this type of service concrete of high quality is necessary; defective work soon becomes apparent; consequently, every effort is made to secure concrete of uniformly high quality. In concrete products an entirely different type of mixtures and materials must be used.

Improvements in Field Methods.—Various types of machinery made their appearance very early in concrete work. Machine-mixing has almost entirely displaced the earlier hand-mixing. There has been a marked tendency during the past twenty years toward the use of mixers of larger capacity. For road work mobile mixers of large size are common.

There have been marked improvements in the direction of a more general practice of washing and sizing of aggregates for all important concrete work.

Central proportioning and mixing plants, particularly for concrete road construction, are a development of the past few years.

Significant features of recent origin are the weighing of materials for concrete batches, and the inundation method of measuring sands in order to secure a uniform quantity in each batch.

One of the most significant developments in recent years is the rapidly growing practice of carrying out field tests on the concrete during construction operations. The widespread present interest in this subject is indicated by the four papers and committee reports on field tests before this convention. Tests recently completed through the co-operation of the Joint Committee on Concrete and Reinforced-Concrete and the contractors have demonstrated the value of field control and have shown that concrete of uniformly high strength can be produced.

The advances in knowledge of the properties of concrete, and the dissemination of this information in this country are an excellent example of co-operative endeavor extending over a period of years. A great many individuals and organizations are responsible for the improvements which have occurred in the art of making good concrete. The technical and engineering societies, the trade organizations, the technical press and the various joint committees have all borne an important part in this development.

The Future.—It may be stated that future developments in concrete practice will probably be along the following lines:

(1) More widespread dissemination of a knowledge of and appreciation of the value of principles of concrete-making based on carefully conducted tests.

(2) Better informed engineers, inspectors, and foremen.

(3) More tests of concrete during construction.

(4) Improved devices for handling materials and for proportioning batches.

(5) Concrete of uniformly higher strength and better quality.

CONCLUSION TO ANNIVERSARY SESSION.

BY RICHARD L. HUMPHREY.

IT now becomes my duty to bring to a close this very interesting memorial session.

It is apparent from what has been said to you today that the Art of reinforced-concrete is quite young. It is less than three-quarters of a century ago since the first experiments on embedded metal in concrete were made. It is less than half a century ago that the first house was built and a little over a quarter of a century ago since more pretentious structures were undertaken.

This is indeed a short period of time as contrasted with the life of ancient Roman structures, surviving remnants of which tell the story of mortars and concretes used more than 2000 years ago.

The speaker is firm in his belief that the ancient lime or cement is not in the same class with the present-day Portland Cement. What a wonderful record the surviving structures would tell of the ancient art of building construction, if a material having the durability of the present American Portland Cement had been used.

The century that has elapsed since John Aspdin obtained his patent for making Portland Cement is but a brief space as measured by human accomplishments, and yet in that time there has been a tremendous development of the use of this material in building construction. It is not improbable that the consumption of cement in this country during the past year was greater than all that was ever used in Imperial Rome.

The great future of concrete lies in its intrinsic qualities as a building material and in the ingenuity displayed in devising mechanical appliances by which it may be economically mixed and placed.

In the nearly twenty years that have elapsed since this Institute had its birth the increasing knowledge of the properties of concrete and its application has been reflected in the transition from heavy massive structures to lighter and more artistic ones. The early conditions in this country surrounding the use of concrete, namely, cheap materials and relatively high cost of labor made it economical to use materials and save the labor of form

construction which would have been necessary in reducing the mass. In Europe the conditions were just the reverse, and the effort was to economize in the use of costly materials; concrete was eliminated where not structurally required and there resulted lighter appearing and more artistic structures. The greater care in mixing and placing concrete and a greater freedom in design led to the erection of structures in Europe which American conservatism would not permit.

The development of the American Concrete Institute had kept pace with the growth of the industry and as the latter acquired stability and permanence the Institute has also acquired stability and a standing that places it beyond that critical period when its continued existence was in doubt.

To those pioneers who guided its earlier destinies, its present position is most gratifying; and the relief felt has all the zest that one experiences in the winning of a hard race.

When the Institute shall have acquired the venerability that one hundred years of honorable achievement will confer, what a great privilege it will be for its members to review the marvelous progress of the industry, in the light of the constructive work shown on the Institute records.

In the ages to come when the members of this worthy Institute learned in the art of concrete construction ponder over the sins of omission and commission of us, their predecessors, just as we are wont to ponder over the art of the ancients, our predecessors, in the use of masonry cementing materials, will they not consider our efforts crude, just as the art of the ancients seems crude in the light of our meagre knowledge of that art.

If we could look beyond the veil of time into the future, what a vision we might behold; Aladdin's lamp would have nothing on the magic that has been wrought by Portland Cement. The tremendous increase in the consumption of Portland Cement is beyond the dreams of the most sanguine and the end is nowhere in sight.

Bold indeed is he who would foretell of the uses of Portland Cement.

One evening while pondering over the probable future development of Portland Cement, the speaker became unconscious of his surroundings and it seemed as if he had attained the long-sought millennium; and as there unfolded before him the wonderful pano-

rama of the future he beheld the City Beautiful of Concrete, artistic, unburnable, indestructible.

The problem of our day had been solved—statuary and other works of art formerly carved from perishable marble were molded in concrete and had a texture equal to that of any stone—capable of taking an enduring polish. This new material, manufactured by scientific formulas, resulting in a uniform standard material free from the unsightly flaws of stone formed by the less scientific processes of nature.

Lumber had been replaced by boards, posts, joists and other structural forms molded from reinforced-concrete of lightweight aggregate, a lumber into which nails could be driven, having strength greater than wood, with the highest resistance to fire and to the action of destructive agencies. This concrete lumber had also replaced wood for door and window sash and other parts of a house formerly built of wood.

Barns, silos, water tanks were all built of concrete; the live stock was fed from concrete feed troughs or feeding floors.

Barges and ships; freight and passenger cars; battery cells; radio, telephone and telegraph poles and masts; sewer and water pipes, drains, conduits and culverts; retaining walls, bridges and quay walls; pavements and curbing, were all built of concrete.

The movement for decentralization of congested centers of population had become effective and with the marvelous facilities for communication, travel by air, radio, wireless, telephone and telegraph, the need for dwelling in thickly populated centers had disappeared. By reason of this decentralization and the use of concrete in building construction, the losses from fire and the burdensome fire insurance had become almost negligible.

The lightweight aggregate reinforced-concrete had opened up new fields of use; its elastic properties and strength replaced the formerly used lumber and steel; its resistance to fire and its durability and the minimum expenditures for maintenance had made it the building material par excellence and had resulted in its use for purposes not heretofore contemplated. Cinder partition blocks and slabs had replaced the materials formerly used because of their insulating value, fire resistance and the facility with which they could be applied.

The researches that had been inaugurated both in Europe and in this country had developed a cement that was as superior to Portland Cement as it had been to the ancient Roman Cement. This new material offered a maximum resistance to sea water and to alkali action.

The elastic properties of concrete and the laws of proportioning and mixing it had become so well understood that its structural application was bold beyond the wildest present-day fancies.

Graceful, artistic bridges of verey great span were now built of concrete.

Concrete had become the building material.

The people lived in concrete houses, walked or motored on concrete pavements; were transported in vehicles of concrete over roadbeds on reinforced concrete rails supported on concrete ties; over concrete bridges, through tunnels lined with concrete; water and liquid wastes were carried in concrete pipes and conduits.

This new cement had a more uniform strength, was practically of constant volume and had a chemical composition which enabled it to attain a very great strength in twenty-four hours. The resulting concrete attained a compressive strength in forty-eight hours of more than 10,000 lbs. per square inch; its toughness gave it a high resistance to shear and bending.

Indeed so marvelous was the development in the quality of Cement that the speaker again became conscious of his present surroundings and the vision of the future vanished.

And what can the speaker say more—a new and wonderful era lies before the Institute.

The present art of making Portland Cement is an evolution of more than 2000 years of effort; the principle of hydraulicity if not discovered, for there is nothing new under the sun, was at least restated at the beginning of the 18th Century.

It is highly probable that the research now under way will lead to the discovery of fresh facts and will result in improved methods of manufacture, producing a cement of greater uniformity as to comparative strength, constancy of volume and binding qualities.

If the mortars and concretes of the ancient Romans have survived the tests of time what may we not expect of the superior Portland cement mortars and concretes of the present and of the future.

The present day concrete is relatively a new structural material; the facility with which it may be reinforced with metal anywhere in its mass to resist tensile stresses gives it a wide applicability.

The protection afforded the embedded metal from corrosion is practically perfect; its resistance to fire where proper aggregates are used is superior to that of other building materials; its artistic appearance when properly designed; its durability; its economy in cost of construction and maintenance are qualities that make it enduring and give it a premier position among building materials.

The development of standard specifications has done much towards securing a uniform material; investigations have been made and research undertaken for the purpose of determining the constitution of Portland cement, yet the field tests of the concrete in which such material is used show a range in value which may in part be due to workmanship and in part to the materials used and yet there may be other factors contributing to these variations.

The speaker looks for a very great improvement in the field methods of making and placing concrete.

Perhaps the doctrine most hurtful to the progress of concrete was that concrete could be mixed and placed with common labor and, therefore, cheaply; doubtless many of the early failures resulted from and investigations showed that in most cases they were due to inexperience and careless workmanship.

The time is not very far distant when it will be found most economical to employ high grade skilled workmen. This tendency has been growing for some time and its need is all the more apparent if one contrasts the conditions prevalent at the time the Institute was formed with those of the present time. It was common practice to mix by hand methods. The field work was in charge of carpenters and foreman; there were few skilled workmen for the reason that there were few reinforced-concrete buildings.

With the acquisition of a more exact knowledge of its properties concrete faces a future of enduring success.

In conclusion may the speaker be permitted to paraphrase a well-known quotation,—

> "all passes, art alone endures—
> concrete outlasts the throne."

Papers Presented at the Regular Sessions of the

Twentieth Annual Convention

of the

American Concrete Institute

February 25, 26, 28, 1924

HOW STRUCTURES WITHSTOOD THE JAPANESE EARTHQUAKE AND FIRE

By H. M. Hadley*

At noon on Saturday, Sept. 1, 1923, there occurred a great earthquake in the east central portion of the main island of Japan, which earthquake in its destructive consequences, is one of the worst recorded in human history. The earthquake itself wrought tremendous instant damage. Structures of all kinds that were deficient in foundations or rigidity broke and fell, bridge piers were overturned, tunnels were blocked, landslides swept down mountains, many thousands of people were killed, and vast havoc was caused. This was but the beginning, however. Occurring at the hour of noon when dinner was being prepared in many houses in Yokohama and Tokio, the collapse of weak structures; particularly the collapse of light-framed wooden Japanese houses, started simultaneous fires in many places, the building department of Tokio reporting that 74 separate and distinct fires started a few minutes after the first destructive shock with a total of 140 original fires, of which 15 were explosive, 85 from stoves (hibachis) and 40 from sparks. Water mains and supply lines had been broken, many streets were choked with debris, fallen wires, abandoned vehicles, etc., and the densely built up cities consisting principally of wooden construction interspersed with brick buildings, and less frequently with buildings of reinforced concrete or structural steel frames, practically all without fireproof openings or details, stood as so much fuel for fire. Fire came. Conflagrations swept the cities, practically wiping out Yokohama, destroying about 50 per cent. of Tokio, including the major part of the business district, and consummating the great disaster.

No exact estimate of the damage is possible. One estimate gives 80,000 human beings killed and 140,000 missing. Of the missing undoubtedly many thousands were killed. Property of vast value was destroyed, probably four to five billion dollars worth. There is the further loss that cannot be evaluated of demoralization of business and loss of records, etc., etc. The Japanese Empire was dealt a very heavy blow.

The Japanese islands constitute a region in which earthquake activity is more frequent and pronounced than anywhere else in the world. Hundreds of quakes occur annually, although the great majority are so slight as to be almost imperceptible. At varying intervals in past history, have occurred earthquakes of magnitude equal to or greater than this one, but this earthquake is the first in Japan to severely test modern building construction.

*District Engineer Portland Cement Association, Seattle, Wash.

Various observations at different stations agree in locating the origin of this present earthquake in Sagami Bay. Whether the cause be volcanic or tectonic, a great buckling of the earth's crust occurred at this point. Preliminary soundings taken between Oshima Island and the southwest shore of Sagami Bay, reveal a 300 ft. uplift in the bottom of the bay at what had formerly been the point of maximum depth, accompanied by a 600 ft. settlement, below its former depth, of the bottom on both sides of this newly uplifted portion.

Incompleted surveys show marked changes in the elevation of the shores surrounding Sagami and Tokio Bays. Extensive uplifting of 12 to 15 ft. has occurred in places with less uplift or sometimes settlement elsewhere. The tremors and vibrations set in motion by these great crustal changes are what constitute the disastrous earthquake of Sept. 1, 1923.

Regarding the character of destructive earthquake vibrations, the following is condensed from a paper written by the late Dr. F. Omori. The complete earthquake motion is composed of several sets of waves of different transit velocities and different lengths of vibrational period. Due to the more rapid dissipation of energy in the waves of shorter period, it depends on the distance of the observing point from the origin of the quake, which of the several sets of waves is the most pronounced and produces the maximum vibration. Maximum waves from a distant origin have an average period of 20 seconds; from a comparatively near origin (the example given has a distance of 2070 km. = 1286 miles), a period of 8 seconds; and from a near origin, a period of from 1 to 2 seconds; generally being 1.5 seconds. Waves of the first two classes are imperceptible without instrumental aid. Waves from a near origin and of short period are classified as longitudinal and transverse, the former and more rapid moving directly in the line from the subterranean origin to the point of observation; the latter and slower moving normal to the plane determined by the point of observation, the origin, and the epicenter, i. e., the point on the earth's surface vertically above the origin.

It is the period between the arrival of the longitudinal and transverse waves—that is, when only the longitudinal waves are felt—that constitutes the preliminary and minor phase of an earthquake. It is the transverse waves, which have in general, only a small vertical component, that produce the shocks of maximum intensity. Even in districts that have been situated directly above the origin of severe earthquakes, the evidence shows much greater damage from the transverse or horizontal vibrations than from the longitudinal, which in any epicentral zone are vertical. Observation shows that for a given period, the amplitude of vibration of the maximum transverse wave reduces inversely as the square of the distance from the origin, or even more quickly, depending upon the character of the intervening material.

From another of Dr. Omori's papers, published in 1900, the following is *quoted:* "In ordinary cases the vertical component of earthquake motion is much smaller than the horizontal. Thus in the severe Tokio earthquake of June 20, 1894, the strong motion seismograph in the

Seismological Institute, recorded a maximum horizontal motion of 73 mm. (period 1.8 seconds) while the maximum vertical motion was only 11 mm." * * * * "I may here note that earthquake motion, though sometimes very violent, is continuous and does not consist of isolated jerks or shocks. The idea prevalent among certain engineers that in destructive earthquakes, buildings are first uplifted by the vertical motion and are then destroyed by being suddenly thrown downward, is quite erroneous."

For the determination of the dynamic effect of earthquakes upon structures, the customary seismological measurements of time and 3–dimentional movement forms the basis. The unknown, unmeasured quantity is the velocity of motion at different points in the path of the earthquake wave.

The diagram herewith shows a graphical representation of one complete earthquake wave. Actually the points 0 coincide, but they are shown separated to illustrate the assumption which is made: namely, that in

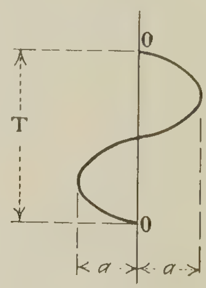


FIG. 1.—THEORETICAL EARTHQUAKE OSCILLATION.

going from 0 to 0 the motion is harmonic; that the velocities along the path vary as the sines of the angles formed by a point moving at uniform velocity around a circular path in the period T. Under this assumption the maximum velocity of the earthquake wave occurs at the points 0, while the maximum acceleration occurs at the extremities of the path. Whence, mathematically,

$$\text{Max. Acceleration} = \frac{4a\,\pi^2}{T^2}$$

and F, the dynamic earthquake effect, the earthquake force on structures =

$$\left(\frac{4\,\pi^2}{g}\right)\ \left(\frac{a}{T^2}\right)\ \left(W = \text{weight of structures.}\right)$$

On September 1 at the Seismological Institute at the Imperial University, Tokio, the most severe shock had an amplitude of 10.3 cm. (4 m) in a period of 1½ seconds, of which the dynamic effect was, according to the above formula almost exactly 1/11 W. Greater motion occurred in later shocks, approximately 6 inches, but the time interval was greater and the first named shock was the maximum in so far as its effect on buildings and

structures is concerned. This observatory is situated on the high firm ground of the city where the motion was not so great as in the low lying business area. The observatory's estimate of the motion in the low lying ground is from two to three times that at the observatory, and they estimate that the severity of the shock at Yokohama was three to three and a half times that at the University.

If a glass of jelly be turned into a dish and the dish be shaken gently, relatively great motion of the jelly will ensue. This behavior is illustrative of the effect of earthquake vibrations in firm and soft ground. The

FIG. 2.—SUMITOME WAREHOUSE—TOKIO.

West and South elevations. 180 x 200 ft. high. Filled with heavy storage valued at $4,000,000 at time of earthquake.

wave amplitude in firm material may be increased several-fold if the overlying ground be soft, spongy or marshy. This fact has an important bearing on the Yokohama-Tokio disaster.

These cities are both situated on the west shore of Tokio Bay; Tokio at the head of the bay and Yokohama, 18 miles distant, at the head of deep water navigation. From Yokohama southward the low irregular hills rise steeply from the water's edge, in some cases cut by waves into bare abrupt bluffs, one hundred or more feet high. These hills are of recent geologic origin consisting of horizontally stratified beds of clays, sands, and gravels, and combinations of these materials, not yet solidified into rock, and topped by a loosely compacted reddish-yellow earth of probably volcanic origin. From Yokohama northward to Tokio, the hills rise less

FIG. 3.—REINFORCED-CONCRETE WAREHOUSE AT HIGASHIKANAGAWA.
Wreck due to foundation settlement.

FIG. 4.—CASCADE BREWING CO.—TSURUMI.
Reinforced Concrete. Only minor damage done to main building shown in photograph.

abruptly and more gradually to the higher levels behind and except at a few points do not lie close to the shore, since the silting up of the upper bay by the discharge of several rivers and other geologic processes have built up a low flat marginal plain of varying width between the hills and the present water's edge. Extensive areas have been filled and reclaimed at various times at Tokio, Yokohama and intermediate points, and most of the destroyed areas in both Tokio and Yokohama were situated on this low-lying ground. Particularly in Tokio, the boundaries of the fire-swept districts closely coincided with those of the low-lying ground on which the great majority of modern buildings were located.

FIG. 5.—REINFORCED-CONCRETE GRAIN ELEVATOR AT HIGASHIKANAGAWA.

In this stage of construction when earthquake occurred. One of the largest reinforced concrete structures in earthquake zone. Practically undamaged. Good pile foundations.

While the earthquake in some instances damaged certain buildings and left other nearby ones untouched, this is not, as is sometimes suggested, to be regarded as a vagary in the occurrence of the shock. In any given district where soil conditions were identical throughout, it is practically certain that the shock was everywhere of consistent performance, and differences in damage are to be attributed to differences in the structures themselves. Also the speed of propagation of the earthquake motion is so great that except as results of local fissuring and cracking of the ground, no differential shaking of buildings can be supposed, i. e., any structure would be affected equally and simultaneously in all its parts.

Building construction in Japan is represented by structures of all characters, those of the type of the ancient empire, shut off from intercourse with the outer world, of course, predominating, but modern structures of steel and reinforced concrete are numerous and are being built in ever increasing numbers. Reinforced concrete had been used in Tokio for some fifteen years; structural steel about five. Brick had been used extensively for many years in the construction of stores, dwellings, factory buildings, etc., while wood and stone masonry had been used for centuries and it was in these last named materials that Japanese architecture had found its own characteristic and individual expression.

FIG. 6.—PHOTO SHOWING GENERAL DESOLATION AT YOKOHAMA.
Russo-Asiatic Bank Building.

Common to all types of buildings is the necessity for good foundations. Without full and adequate support no structure could be expected to stand and no structure did. There has been considerable discussion regarding the merits of mat and pile foundations. Either is satisfactory so long as unyielding support is obtained. Inasmuch as the severe earthquake motion is predominantly horizontal, the cushioning effect which it is claimed can be obtained with mat foundations is a cushioning not in the direction of the severe forces. On the other hand, where mat foundations do not have piles beneath them carried to a depth sufficient to furnish full and unyielding support, very expensive damages may result from the bodily settlement of these mat foundations with their superimposed

building into the soft ground under the shaking effect of the earthquake. Where individual footings are used they should be connected with beams of sufficient strength to maintain them at all times in their normal relative positions.

Reviewing the behavior of structures of the several building materials under the test of earthquake and fire, the following summary can be made:

1. *Wood.*—In the fire-swept areas all evidence was destroyed. Elsewhere the performance was generally good. In Japanese houses where heavy tile roofs were carried by light wall posts with little bracing, more

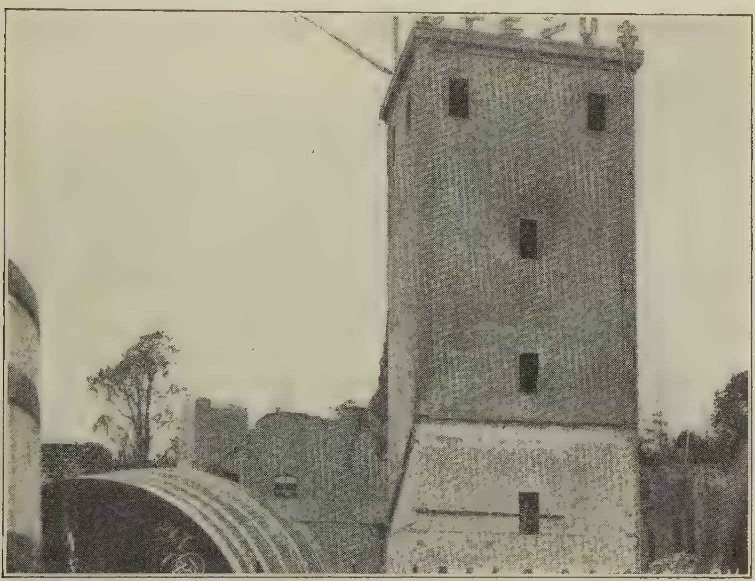


FIG. 7.—KIRIN BREWERY—YOKOHAMA.

Photo shows undamaged reinforced concrete tower used for malt and hops storage.

or less racking with consequent cracking of plaster ensued and, of course, there were numerous collapses of buildings inadequately braced and stiffened. The Japanese are skillful carpenters and their houses are framed with mortise and tenon joints and wooden pins, etc. The combination of great strength with light weight that is the inherent character of wood, is most advantageous in structures subjected to earthquake shocks. At the same time the problem of efficient joints and connections becomes increasingly important. Two of the finest examples of wood construction are to be seen at an industrial plant at Kawasaki, where two buildings of reinforced concrete collapsed and a third was badly damaged. These buildings had been designed by an American architect who had an adequate concep-

tion of the forces to which these buildings might be subjected. Consequently all connections between beams and the columns which were closely spaced in one direction, had been knee-braced and well bolted and washers of generous size were used. At each of the three floors the buildings were trussed horizontally at all four walls and at all four corners vertical trusses in the plane of the side walls extended from the foundations to the roof. These buildings, full of valuable machinery, escaped without the slightest damage.

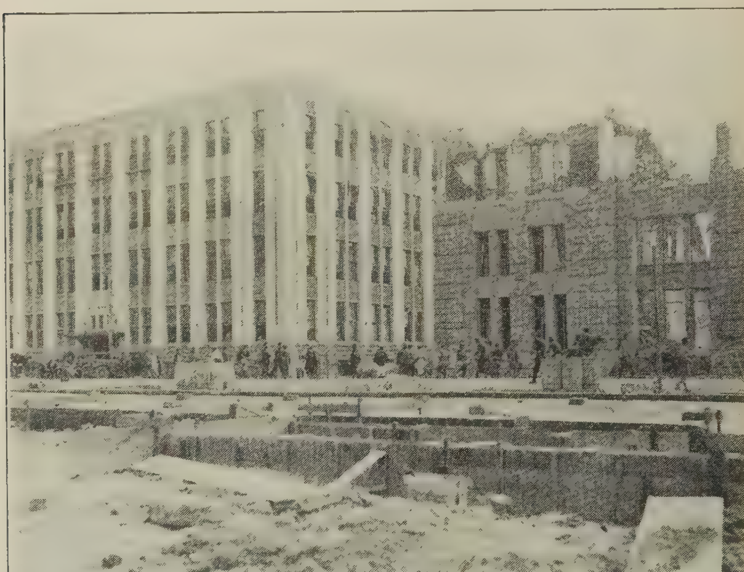


FIG. 8.—ABSOLUTELY UNDAMAGED REINFORCED-CONCRETE BUILDING ON BLOCK IS MARUNOUCHI CENTRAL, TOKIO.

2. *Brick.*—Regarding brickwork, it should be stated at the outset that practically all construction is with Japanese brick, of a strength no greater and probably weaker than American common brick. Mortar undoubtedly varied greatly in quality but the execution and workmanship of laying up brickwork is probably superior to American. In the vast amount of shattered brickwork fractures in general occurred through the brick itself and not along the mortar joints and also revealed the inner mortar joints well filled and without voids or holes. Construction of buildings generally in Japan is a more slow and deliberate process than in America, and workmanship can be correspondingly more thorough and careful.

With certain splendid exceptions, brick failed badly and was responsible for great loss of life. The following is from the report of K. Satch, Structural Engineer of the Tokio Building Department, who was in charge

of the investigation of 485 brick structures in Tokio, 49 of which were situated on the relatively firm higher ground, and 436 on the low lying ground. Needless to observe, the percentages based on the low ground buildings are more representative than the high ground, since nine times as many buildings are under consideration.

Districts.	Entirely Collapsed.	Partially Collapsed.	Heavily Damaged.	Slightly Damaged.	Undamaged.	Total.
Yamanote (hard soil)	3	9	15	16	6	49
Shitamachi (soft soil)	44	104	87	120	81	436
Showing the above figures by percentages:						
Yamanote	6.0	18.4	30.6	32.7	12.3	
Shitamachi	10.0	23.9	20.0	27.5	18.6	

FIG. 9.—TEMPORARY QUARTERS OF YOKOHAMA SPECIE BANK.
Reinforced concrete and brick filler walls. Damage due to brick filler walls.

Fundamentally the weakness of brick masonry is its low tensile strength which is insufficient to withstand the bending and swaying that an earthquake causes unless the walls be of considerable thickness and be well stiffened by the floor systems or heavy division walls.

3. *Structural Steel.*—As stated, the use of structural steel for building purposes is very recent in Japan. Nevertheless numerous large structures had been completed and others were in the course of erection, when

the earthquake occurred, and these buildings were sufficiently numerous and varied in character to permit conclusions to be drawn regarding their behavior. It is unnecessary to enumerate the known excellences of structural steel, although the severe property losses that occurred in the burned areas from its use without any fire protection emphasizes the necessity for properly safeguarding it. What is the outstanding and unusual feature of skeleton steel framed buildings under earthquake conditions is their tendency to rock and sway. The sudden quick motion of the ground produces a correspondingly sudden bending and deformation of the steel columns

FIG. 10.—PATENT OFFICE EXHIBITION BUILDING—TOKIO.

Rear North Elevation. Reinforced concrete. Failure due to inadequate foundations causing a 2-ft. settlement.

and the horizontal load is thrown upon the stiffer vertical members of the structure: the exterior walls, interior partitions, stair and elevator enclosures, etc. Whereupon these secondary vertical members, unless they possess adequate power of resistance, are broken and shattered, and thereafter the resistance is furnished by the structural steel alone, when on the other hand the wall construction possessed the adequate power of resistance no damage was entailed, the buildings standing rigid and unyielding. Four large completed steel frame buildings in Tokio and two practically completed escaped without damage from the earthquake. The common characteristic of all of these buildings was their complete or extensive use of reinforced concrete wall construction. These buildings are the Industrial

Bank of Japan, the First Mutual Building, No. 21 Mitsubishi, and the building of Katakura & Company. These were completed and occupied. The two under construction were the Marumouchi Hotel and the Kokko Life Insurance Co. Building. These buildings all escaped with absolutely no damage. They were the only large steel frame buildings in Tokio which did. On the other hand, the 11 other large steel frame buildings which employed brick for their wall construction all sustained more or less damage due to the shattering of their exterior walls, breaking of interior par-

FIG. 11.—INTERIOR VIEW SHOWING BADLY DAMAGED FIRST STORY COLUMN IN BUILDING NO. 25 OF INDUSTRIAL PLANT AT KAWASAKI

Shows absence of spiralling and very light binders.

titions, destruction of marble trim and wainscoting, damage to elevators, etc. One other large building had its structural steel frame completed and concrete wall construction started in the lower stories, but this building was not sufficiently advanced in its wall work to serve as proof of the merits of concrete wall construction for resistance to earthquake. Three other buildings, the Sumitomo Bank, the Mitsubishi Bank, and the main Central Railway Station were of structural steel and heavy brick or stone masonry. These were all rather low structures, however, and are not to be classed with those previously mentioned. Outside of Tokio no large structural steel frame buildings of the above character existed in the earthquake region. A number of industrial plants had mill and shop buildings

of structural steel which behaved in an entirely satisfactory manner when adequately braced and not exposed to fire.

A general survey of the situation leads to but one conclusion: Properly designed structural steel buildings, well braced and thoroughly fireproofed can be made earthquake-proof; the simplest, cheapest and most efficacious bracing can be secured by making the wall construction of reinforced concrete.

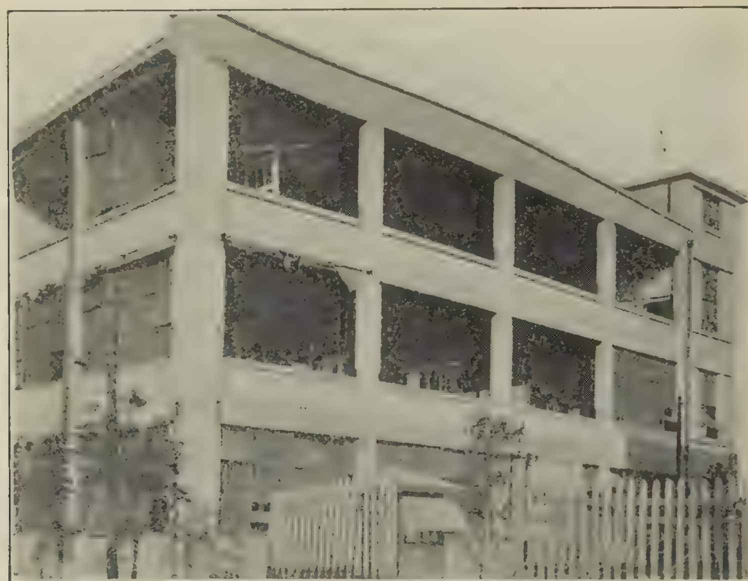


FIG. 12.—BUILDING NO. 18, INDUSTRIAL PLANT AT KAWASAKI, BETWEEN TOKIO AND YOKOHAMA.

Reinforced concrete. Damage caused by poor foundations and lack of rigidity.

4. *Reinforced Concrete.*—The performance of reinforced concrete under the test of earthquake and fire can only be classed as highly satisfactory. A survey of reinforced concrete construction in Tokyo Prefecture conducted by the Tokyo Building Department under the direction of Mr. Y. Nagata, Chief Engineer, resulted in the following findings:

	Entirely Collapsed.	Partially Collapsed.	Greatly Damaged.	Partially Damaged.	Undamaged.	Total.
	8	11	42	69	462	592
Percentage of total	1.3	1.9	7.1	11.7	78.0	100.0

I questioned Mr. Nagata closely regarding the exact meaning of the terms of his classification, that are perhaps indefinite. Regarding "Entirely

Collapsed" and "Partially Collapsed" there can be no uncertainty. "Greatly Damaged" he illustrated by two buildings: The Hoshi Drug Building and the Industrial Club of Japan. "Partially Damaged" means without severe damage to structural frame, although the walls may be shattered. He gave me as examples of this classification the following structural steel framed buildings, Tokyo Kaijo, N. Y. K. Maranouchi, and Yuraku. "Undamaged" includes cases of minor cracking of walls, but no damage to structural frame. The Sumitomo Warehouse illustrates this classification. As stated this survey includes damage by fire. Warehouses classed as par-

FIG. 13.—POWER PLANT AT HIGASHIKANAGAWA.

Reinforced concrete boiler house. Failure due to faulty engineering design. Heavy roof was not anchored, tore free from the columns and fell.

tially collapsed would shift to the undamaged classification were the earthquake alone considered and there are several other buildings which would be similarly changed.

Elsewhere in Tokyo Prefecture the results obtained were as follows, this damage being solely by earthquake:

	Entirely Collapsed.	Partially Collapsed.	Greatly Damaged.	Partially Damaged.	Undamaged.	Total.
	8	9	7	5	89	118
Percentage of total	6.8	7.6	5.9	4.2	75.5	100.0

In giving me this information, Mr. Nagata told me it was preliminary, and subject to change and correction, but substantially was correct; changes to be minor only.

Considering all things, this is a highly satisfactory performance. Particularly is this true when the character of concrete in this district of Japan is known. No criticism can be made of the character of Japanese cement used in this district, but their aggregates as used would be summarily rejected in any construction work in this country. The sand itself is quite fine, of uniform size and of indifferent structural quality. The

FIG. 14.—MITSUI NO. 3 BUILDING—TOKIO.

North and West elevations. Reinforced-concrete—110 ft. high. Gutted by fire—uninjured by quake. Debris in foreground from adjoining brick structures. Concrete stack at left on Mitsui No. 4 Building.

greatest trouble results from the coarse aggregate. This is river gravel of fair strength which is dug by hand from river bars. It is used in the condition in which it is dug, all particles being coated with silt, and with an admixture of about 25 per cent sand in the gravel. A composite sample taken from six different building jobs under construction in Tokyo showed the following results: Gravel contained 1 1/10 per cent silt in the coatings; sand contained 1½ per cent silt; organic plate No. 2. Seven-day briquettes 85 per cent of Ottawa; 28-day briquettes 89 per cent of Ottawa. Average compressive strength of six 4 x 8 cylinders stored 28 days in damp sand at 70 degrees F., 1103 pounds per square inch. With a division of

FIG. 15.—POWER PLANT AT OIMACHI.

FIG. 16.—NICHI NICHI BUILDING—TOKIO.
Reinforced concrete. No damage.

material on the $\frac{1}{4}$ inch screen the 1: 2: 4 mix which they so extensively employ would be actually a 1: 3: 3. With excess of water and poor curing it is unlikely that their building concrete possesses a strength more than 800 or 900 pounds per square inch. Nevertheless, it is concrete of this character which performed as the above surveys show. It can only be expected that with really good concrete, a far better record would have been established.

FIG. 17.—JUTSUGYO BUILDING.

South and West elevations. Reinforced concrete, veneered with Japanese tile. Had recently been completed at time of earthquake.

Causes of Failure.—The only good that can result from a disaster of this character is the lesson that is to be learned from it. In the field of concrete where failures occurred they were due to one or more of the following conditions:

1. Inadequate foundations; 2. Violations of commonly accepted principles of engineering design; 3. Lack of rigidity in buildings. The third cause is the feature that is peculiar to earthquakes. The small one story market building at Yokohama shown in photograph No. 400 was 75 x 300 ft. in plan with 25 ft. square bays, was unusually well built, was thoroughly reinforced and the reinforcement was all fully anchored and hooked and no sign of foundation trouble was to be seen. Nevertheless, lacking

rigidity, lacking resistance to the bending stresses developed by the sudden horizontal motion of the ground on which the structure stood, it failed. How this stiffness is to be obtained is the problem of earthquake-proof construction. The solution is to make part or all of the wall construction of

FIG. 18.—OKURA & CO., LTD., BUILDING, TOKIO.

Reinforced concrete—veneered with Japanese tile and stone.

reinforced-concrete integral with the columns. This is but another way of saying to increase the dimensions of certain columns, since the introduction of a duly designed reinforced-concrete wall between and integral with two columns makes them act as one column, with a resistance to bending many times greater than that which the two separate columns formerly possessed. For example, a row of ten 1-ft. square columns 10 ft. on centers would

have a resistance to bending in the direction of the row which is a function of the square of the depth of the columns and may be represented by 10. The introduction of a reinforced-concrete wall between two of these columns would make a single column of the two, which single column would have

FIG. 19.—KASHIMA BANK BUILDING—TOKIO.

Reinforced concrete. Burned by fire. Undamaged by quake.

a resistance to bending equal to 11^2 or 121. To secure equal resistance by increasing the size of each individual column would require that they be made 3 ft. 6 in. each.

The one story market building of photograph No. 400 had open walls between the wall columns. The introduction of 10 ft. wall sections at all four corners—points where the sacrifice of light is a minimum—and of

small sections of wall perpendicular to the length at one or two points, would have unquestionably saved this building. How much wall is needed for any design is simply a question of the weight of the structure and the earthquake forces, which acting in any direction, must be designed for

FIG. 20.—MITSUI NO. 4. BUILDING.

Reinforced concrete.- Gutted by fire. Undamaged by earthquake and does not have a crack in it.

Where, as in certain buildings, it may be undesirable to use walls, then equivalent stiffness must be developed by trussing or by frame action. The use of relatively small diagonals for tension members in conjunction with the regular beams and columns of a building, would accomplish the desired result with a maximum of opening.

Regarding the special features of Japanese reinforced-concrete design, the most noteworthy is the care given in their best practice to adequately anchor all reinforcement. Footing stubs and column verticals are hooked at the splices and all possible anchorage for all bars is secured. With the reversal of stress that accompanies earthquake motion the advisability of this practice is manifest. Laps of column verticals are based on tension, not compression. An attempt to vary and stagger the plane at which horizontal construction joints in columns are made so as not to have all

FIG. 21.—MITSUBISHI HEAD OFFICE BUILDING—TOKIO.
Reinforced concrete—tile and stone veneer. Undamaged.

joints in the same plane, was also observed. I question the necessity of this in a structure of good concrete with joints kept free from laitance, wood and general dirt.

Wire mesh was used quite extensively for floor slab reinforcement. I noted that it was invariably in the bottom of the slab at supporting beams instead of at the top where it belonged. Whether this was intentionally or unintentionally misplaced, I do not know.

The use of metal lath between beams for combined forms and slab reinforcement was observed in two failures. This construction provides no continuity of reinforcement in beams, and there is a lack of strength in consequence.

The omission of slab reinforcement in a concrete joist job was likewise noted. This detail did not increase the strength of the structure. The omission of mesh reinforcement in fireproofing structural steel columns entailed disastrous consequences, in one or two instances, most strikingly in the case of the Mitsubishi warehouse at Tokyo. This was a large two-story warehouse of reinforced-concrete except for the interior columns which were angle latticed steel, fireproofed with two inches of concrete but without mesh. This building withstood the earthquake without any damage, but in the ensuing fire the concrete fireproofing spalled, the columns buckled, and the major portion of the building collapsed.

Of all structures of reinforced-concrete, chimneys gave the most unsatisfactory performance. There were many that successfully withstood the earthquake, but there were likewise many which failed, and in several instances caused great damage to structures nearby. The poor quality of the concrete revealed itself most plainly in these structures, where concrete of especially good quality is required.

Bridges of structural steel and reinforced-concrete gave entirely satisfactory performance when they received the support they deserved from piers and abutments. The 125 ft. skew arch on the Tokyo elevated railway, a short distance north of the Central railroad station, was entirely undamaged. Several steel bridges in the fire zone suffered damage to their floor systems where wood decks were used and wooden trestles likewise suffered damage from fire. One of the most disastrous failures was on the main line railway at the crossing of the Baniu River where a long steel plate girder bridge went down due to the overturning of brick masonry piers. Mention should be made of the admirable performance of the elevated railway construction in Tokyo. South of the station for two or three miles the track is elevated and carried on a series of brick masonry arches of 40 to 50 ft. spans, terminating in heavy abutments at the street intersections. This work stood perfectly. So likewise did a mile of reinforced-concrete elevated construction, north of the Central station.

Concrete retaining walls gave generally satisfactory performance. Foundation troubles were responsible for any damages which I observed.

Regarding tunnels, the only lined tunnel which I saw was through the Bluff at Yokohama, about ¼ mile long and was lined with brick. One portal was damaged by a landslide, but otherwise it escaped undamaged. On the main railroad lines through the mountains landslides blocked numerous tunnels.

In conclusion I would state that in general satisfactory performance was obtained only with structural steel or reinforced-concrete; that the concrete was of uniformly inferior character; that despite this fact its behavior was admirable; that skeleton construction with either material without bracing is inadequate; that firm and unyielding foundations are essential; that in building construction the use of reinforced-concrete walls is the simplest, best and cheapest insurance against earthquake damage.

THE EARTHQUAKE IN JAPAN.

By Joseph S. Ruble.*

The first earthquake shock on September 1, 1923, was at 11.59 a. m., the second about ¾ of a min. later, the third about 3½ min. later, and the fourth 11 min. later. The maximum intensity of the shock resulted in a horizontal motion, as recorded at the Imperial University in Tokyo, of 5½ in. Their devices for recording vertical motion, built to withstand the severest kind of earthquake, were destroyed and as a result they have no record of the vertical motion. During the first 12 hours we experienced a total of 222 shocks. During the first 17 days 1319 shocks occurred. Anyone on the ground would have said that the number was even larger.

Cause.—The cause of the shock, as reported by experts, was a settling of the sea bottom of Sagami Bay. The reports which I have state the depth of the sea at these points prior to the earthquake as being approximately 5,000 ft. The reports concerning the amount of settlement vary considerably. It is safe to say, however, that the settlement was somewhere between 300 and 600 ft. in depth covering an area 5 miles wide and approximately 15 miles long.

I feel there is little question but what the shock was very much more severe in Yokohama than it was in Tokyo. As stated before, the horizontal motion of the earth's surface in Tokyo was recorded as 5½ in. It is a safe assertion that the horizontal motion of the earth's surface in Yokohama was nearly twice this amount.

The area affected by the earthquake is generally included within a radius of 100 miles from the point of origin.

Damages.—It is, of course, impossible to state the actual loss as a result of the earthquake. The Japanese authorities have estimated their loss of property at $900,000,000 or more. This is equivalent to 2 per cent of their national wealth. When consideration is given to the loss of production and the increased cost of replacement over the actual cost of property destroyed, one can reasonably feel that the total economic loss will be as much as two or three times the actual loss of property, and when so considered, the figures are rather astounding. I feel that Japan is financially sound; they have a large gold reserve both at home and abroad, and are undoubtedly in position to pay for anything they desire to do. Their credit, therefore, should be considered good.

The loss of life is reported to total 150,000, divided approximately as follows:

Tokyo	80,000
Yokohama	22,000
Elsewhere	8,000
Total unaccounted for	40,000

* H. K. Ferguson Co., Cleveland, O.

It is hardly possible for one who has not witnessed the disaster to realize what these losses mean. It is difficult for one who has witnessed the disaster to realize what they mean. Even a fair appreciation of them, however, would lead one interested in any construction problem in Japan, to try to be very careful of the recommendations he might make in order to avoid feeling later that he may have been responsible for a similar loss of life and property.

The company I represent has in process of construction in Japan a number of rather large buildings, the designs for which were completed prior to Sept. 1. All of these designs have been checked by three engineers experienced on building construction in earthquake territories, and except

FIG. 1.—FOUNDATIONS FOR BUILDING IN JAPAN.

for minor modification and detail, the plans for superstructures are being executed as originally prepared. The foundations we have modified. (Fig. 1) The original plans called for concrete piles surmounted by pier caps to tie each pile nest together, the exterior group of piles for each building being tied together by a spandrel wall. Following the earthquake we thought it advisable to add reinforcing to the piles, and to tie each pier to its neighbors by a reinforced-concrete beam to safeguard the dislocation of one pier from its relation to all others.

Soil.—The nature of the soil on the low portions of Yokohama and Tokyo is so-called alluvial deposit washed down from the mountains, and overlays in general a strata of gravel. In some locations this wash will be entirely mud; in others it will be mixed mud and sand, and again in others it will be fine sand. Its depth varies from 3 to 150 ft. or more. In the background and on the higher areas of these two cities the soil is a red clay beneath which is encountered a soft limestone rock.

In order to operate successfully in this territory one needs, therefore, to be qualified to handle all types of foundation construction.

The angle of repose is questionable, may be only 2 or 3 deg. from either horizontal or vertical.

Foundations.—The Japanese themselves, at least those with whom I conferred, give preference to a foundation on solid soil; second choice to caissons or long piles to hard bearing, and, in general, third choice to short piles in soft soil covered by a heavy reinforced-concrete mat 2 ft. to 5 ft. thick.

Wherever the foundations were weak you can depend upon it there was serious damage to superstructure on Sept. 1. It is an apparent fact that the force of the earthquake is more severe along certain veins of the earth surface than at other points. This can be appreciated when one realizes that the damage in Kawasaki, for example, was extremely severe, while Omori was passed by with very little harm, only to have Shinagawa very severely shaken. Shibaura was then passed by with little damage, and Tokyo was struck rather violently in spots.

There are evidences of foundation failures on hard ground in northwest Tokyo four miles from the water's edge. I have in mind a four-story concrete building, on which the southwest corner was apparently pushed up about 4 in., while the other portions of the building stayed in their original position, the result being that each spandrel girder, including the foundation spandrels to this column was broken on each story, and the corner of the floor slabs on each floor was broken. I, of course, had no bench marks to refer to and do not know that the two corners were actually raised, but either this happened or all the rest of the building settled; but the whole ground floor seemed to be in good condition.

There are a number of foundation failures in buildings where foundations were constructed of long piling driven supposedly to hard soil. On most buildings examined there was evidence of one or more piers having settled. This resulted in overloading of the piers which did not settle, which in turn caused the latter columns to break in compression. I have used the word "compression" here rather reluctantly for the reason that while such columns were undoubtedly overloaded in compression, it is quite possible they would have carried such load were it not for the belief that the earthquake created an effort to revolve the base of the column in a circle while the motion of its top was resisted by the mass of the building. In any case, the defect of unequal settlement resulted in serious damage to many buildings. The other defect implied in the immediate preceding remarks was that a long pile with point bearing, after having been driven through soft soil, was dangerous, in that one pier or group of piles may have their top move around in the soil more than another pier or group of piles, unless they are all tied into a unit by the use of beams or girders to hold each group in its relative position to all others.

Foundations.—Foundations constructed with the use of short piles

covered with a reinforced-concrete mat have some desirable characteristics, but only when the areas to be covered are fairly large. This type of foundation is not good on small areas, because in soil that has any tendency to be liquid the settlement would become dangerous. Where the areas are large, however, there are some advantages to be gained, and even though evidences of settlement were apparent on some of the larger buildings, such settlement was not enough to be called at all dangerous.

I am of the belief that solid ground is shaken more violently than soft ground, rather because of its solidity it transmits to the structures supported by it a greater proportion of the force of the earthquake shock, than is the case with soft soil, which has a tendency to absorb the shock, and which has the further tendency of permitting the shock of the earthquake to pass under rather than into a floating slab, and thereby it seems probable that a building supported on floating foundations, in addition to avoiding unequal settlement, receives less injury than one supported on solid ground.

There is some question in my mind concerning the practicability of either of the two outstanding examples of concrete piles used in America today; one gaining its strength from the principle of surface friction developed by the cone-shaped pile, and the other gaining its strength from the principle of an expanded head to gain increased bearing area. There are plenty of places in Japan where it will be impracticable to drive either of these types of piles to solid bearing, and when one stops to consider the possibilities, based on actual observations, of an earthquake in Sagami Bay causing water to spurt out of the ground in the vicinity of Tokyo, one can appreciate that water so forced through the earth's surface will act on a pile in the same manner as the water jet which we use to drive the pile. That such action will cause the pile to lose its normal strength, and if loaded, to settle with resultant damage to the structure it carries.

Further observation and test is desirable. Meantime a greater factor of safety should be resorted to.

Retaining Walls, Bulkheads, and Abutments.—These can generally be considered under the same heading. In Japan they are constructed for the most part of stone or concrete, with a few examples in brick.

Many retaining walls are constructed by the use of a long stone laid cross-wise in the wall. The stones generally have some taper from the front to the back of the wall. The front face is roughly dressed, and when laid, the top, bottom, and sides of the front face fit neatly against their neighbors. No mortar is used, but dirt and stone is used to fill in the crevices at the back. This type of wall seems to have withstood the earthquake best for the reason that it is able to accept wave motion without breaking. The principal shortcoming observed in connection with this wall was that the stones used were not of sufficient length to resist overturning of the wall in which they were used.

Retaining walls constructed of heavy concrete with vertical joints to permit the wall to accept wave motion came through the earthquake quite well.

Many concrete walls are badly broken for the reason that they were built in long sections. Some of them are broken horizontally wherever a construction joint was made. In building this type of wall care should be taken to cast it to its full height at one pouring of the concrete.

Reinforcing improves the strength of this type of wall, but, in general, it would seem that the reinforcing used was not sufficient to resist earthquake.

Stone walls constructed of large stone laid in mortar have some examples that came through the quake very good. Other examples are badly damaged, particularly by vertical cracks—some by horizontal cracks.

Brick retaining walls act similar to those of stone masonry. The shortcoming in construction applying to all types of retaining walls was the lack of width to be safe against overturning. I know of one stone masonry wall 250 ft. long, the front of the stone bedded in mortar and the back of the stone in dirt; this wall was 20 ft. in height, its greatest thickness was not over 27 in., its slope from the vertical was about ½ in. in 12 in., and amazing as it may sound, only 25 per cent of the length of this wall turned over. This statement is still more amazing when supported by the fact that its length was approximately at right angles to the force of the earthquake.

Abutments.—Built of heavy concrete following our idea of proportion for design, in general, came through fine, principally because of their short length. The main shortcoming of those that failed was apparently a lack of bond between the footing and that part of the abutment above the footing. When built on top of piling the bond to the piles must be good, and the bond between the abutment itself and the pile cap must be good. For example, the railroad bridge at Kozu, a double track structure with 28 piers carrying 60 ft. or 70 ft. steel spans was constructed approximately as follows: Piles were driven. Concrete pier cap was cast on the top of each group of piles. The pier, or abutment, was then cast on top of the pier cap without having made provision for a real bond between the pile cap and pier or abutment. The result on September 1st was to have each pier and abutment shaken off of the pile cap into the river, taking the bridge spans with them.

Stone masonry abutments constructed of large stone doweled are good. When undoweled only fair, and where stone are laid dry without dowels, not safe. I have seen examples of the latter construction where several large stones near the bottom of the abutment worked themselves entirely out of the abutment as the result of the continued vibration of the earthquake.

So-called arched abutments of two or more groups of piles are not safe; one or more of the legs break. The safe method to follow is to build

solidly from pile tops up. On the government railroad properties, generally, retaining walls, abutments, and piers, are well constructed and were only damaged where most severely shaken. Construction of this kind for the most part has been subject to foreign design and supervision.

Private work was not so well done with resulting greater losses.

Fences.—There are numerous examples of stone and brick fences that failed mostly at rigid angles to the force of the earthquake. In Tokyo, for example, the earthquake came from the south and practically every fence that stood at right angles to the direction of the quake fell to the south, and for the most part they failed just at the top of the footing, or turned the footing over with them. They were not reinforced.

Stacks.—Are limited practically to two types: steel shells, and reinforced-concrete. Self-supporting steel stacks, in general, came through without damage. In several instances, however, I have observed settlement of foundations which caused the stacks to lean. On the other hand, a great many concrete stacks did fail and quite a few of those that remained standing have damage of one kind or another. The damage on the concrete stacks has occurred at all points in the height of the stack. In some cases the damage is at the ground line and in others within twenty feet of the top. In general, however, it may be stated that the damage occurred at approximately one-third of the height, some by compression, some by tension. In Tokyo and Yokohama where stacks fell they, in general, fell towards the north or northwest.

Towers.—In passing, it is interesting to note that radio towers and the poles and towers for telephone and transmission lines, with which Japan is covered, were practically unharmed by the earthquake. It is true most of this service was ruined in both Yokohama and Tokyo, but mainly as a result of their being in close proximity to other structures.

Tunnels.—There are examples of brick, stone masonry, and concrete tunnels. To the best of my knowledge there are no tunnels which fell as the result of the earthquake. Some of them lost their portal arches and wing walls, they being carried away by landslides. Apparently a tunnel constructed inside the earth becomes an integral part of the earth and moves with it as a whole and until the earth in which the tunnel is constructed cracks, the tunnel is safe.

Building Superstructures.—I have several tables to show the relative value, rather their order of value, first for safety from quake, second for safety from fire, and third for safety from both earthquake and fire. Based on observations after Sept. 1, the building which I consider best in each class being placed at the top of the list:

Types of Structures (For safety from quake):

1. Structural Steel (Plain or Fireproof)
2. Wood Frame (Mill Type)
3. Concrete
4. Brick and Wood
5. Brick and Concrete Floor
6. Wood Frame (Light)

Types of Structures (For safety from fire):

1. Concrete and Structural Steel Fireproofed
2. Brick and Concrete Floors
3. Structural Steel (Plain)
4. Brick and Wood (Heavy)
5. Wood Frame (Heavy)
6. Wood Frame (Light)

Types of Structures (For safety from both earthquake and fire):

1. Structural Steel Fireproofed
2. Concrete
3. Structural Steel (Plain)
4. Wood Frame (Mill Type)
5. Brick and Wood (Heavy)
6. Brick and Concrete Floors
7. Wood Frame (Light)

Reasons.—I place the safety of human life above all other considerations. The safety of property and invested capital should take second position.

1. Fireproof Structural Steel Building: None fell or collapsed as a result of the earthquake to the best of my knowledge and belief. None of them were destroyed by fire except for the contents and the trim that was in them.

2. Concrete Building: None were seriously damaged as a result of fire, except for the contents and trim. Some failed as a result of the earthquake.

3. Plain Structural Steel: None fell as a result of the earthquake. Some failed and collapsed as the result of the burning of their contents and trim.

4. Heavy Wood Frame: Most of these buildings withstood the quake and permitted occupants to escape. Being slow burning, the fire was of secondary importance.

5. Buildings of brick walls with heavy wood floors are structurally more secure than when a concrete interior is used. Ocupants can escape and fire is again of secondary importance. On this class of building most of the failures were in the walls.

6. Buildings with brick walls and concrete floors had their walls fail by the floors sliding over the walls. The columns also failed by quake. This type of building indicated serious danger from collapse at the time of the quake. None burned except the contents.

7. Light Wood Frame Buildings. Many failed. Many burned. Unsafe from all angles.

The reason for placing the concrete building ahead of the plain structural steel building is based on my belief that the concrete building

can be made reasonably safe structurally by the exercise of proper design and proper workmanship. Some very fine specimens now stand in good condition.

Brick construction can be safeguarded by the use of reinforcing steel.

The light wooden frame building as constructed in Japan can be improved upon by greater care to provide bracing, and by the adoption of a light roof in place of the heavy tile in general use today.

The sprinkler system where earthquakes occur is a questionable safeguard against fire, as immediately after the earthquake you are liable to have no water supply as a result of broken water mains.

Rate of Vibration and Acceleration Are Important.—A certain building may go through one quake with no damage and fall in the next, whose severity apparently is half as great as the first, but whose vibratory rate acceleration happens to be the dangerous one for this particular building.

Walls and Partitions.—The materials to be used for enclosures, roofs, sides, and partitions are quite important; rain, wind, heat, cold, fire protection, strength, weight, and liability to damage by quake must all be given consideration.

In general, it is desirable to minimize weight in such materials as a means of keeping to a low maximum the stresses in the structural design. Design the structure to be safe in itself without the aid of the strength of these materials. Reinforced concrete is the outstanding example of a material that can be used for this purpose, i. e., have its strength used in walls and thereby reduce the expense for the structural design itself. I wonder if it is practical to make a higher strength concrete for such use as a means of minimizing weight. The use of terra cotta tile in walls is not good.

Observations.—Greater care is needed in the design and construction of foundations to make them safe against unequal settlement. This is of first importance and I would like to make it clear that in my opinion one of the fundamental causes of failure was in the foundations.

The most outstanding shortcomings in all building construction is that of the lack of proper bracing. Ann increase in the size of columns, and a greater use of solid wall sections, as a means of resisting the motion of one floor over another, deserves serious consideration.

Defects were apparent in the amount and kind of reinforcing for columns and spandrels, particularly hooping and length of lap. (For safety all rods should have hooked ends.) The cross sectional area of columns, spandrels and beams were questionable. The size of knee braces and soffits were small. There seems to have been a lack of consistent effort to minimize weight at the top. Floors, in themselves, generally were little damaged, and in some buildings not at all, even though every column was either broken or cracked.

In the execution of work the contractor is confronted with a problem to obtain the proper materials for concrete buildings. There are a number of cement manufacturers in Japan. Out of the total two of them

manufacture a good brand of cement. The brands of these two firms will meet the requirements of the American Society for Testing Materials.

Gravel in the vicinity of Tokyo and Yokohama is pretty fair material. Its supply is abundant, but the methods used for obtaining it are pretty crude. In the summer men dig it out of the river by standing in water up to their waist, and hoe the material into a screen which they pick up in their hands and shake under the water, both washing and screening the gravel before depositing it in the boat alongside of them. This boat in which the gravel is transported has been previously towed up the river by man power, against a rather rapid current, a distance of some 5 to 8 miles. After loading the boat floats down the river with the current. In the winter the gravel is taken from gravel banks out of the water, and you are liable to receive unwashed gravel, which, when used, will leave a layer of laitance at the top wherever concrete work is stopped. All construction joints, therefore, are inclined to be weak because of this laitance, and the failure of the contractor to remove it.

Sand in the vicinity of Tokyo and Yokohama is not up to our idea of quality. It is too fine for good work, it is not sharp, and unless one is extremely careful it will have more or less laitance in it on delivery. A very excellent grade of sand for concrete purposes may be obtained at Osaka, a distance of about 300 miles from Tokyo, but it is difficult to convince the average owner in Tokyo and Yokohama that he should pay a premium to obtain material with which he can be assured of a good grade of work.

Construction workmen have their own ideas of detail, but can and do obtain the same results that we obtain here, conditioned only on their being properly instructed and supervised.

In the past the Japanese have greatly favored the use of concrete building materials because they are able to produce these materials for themselves. The earthquake of September 1 has undoubtedly detracted from the good standing of the concrete structure in Japan. This is more or less a result of self-satisfaction or indifference on the part of those interested in the concrete industry.

While Yokohama and Tokyo have building codes, the codes are not adequate for safe building construction. They should have your attention in the way of constructive advice towards the improvement of these codes to meet the conditions required for safe building. The cement industry in general is satisfied to produce a cement, which, when tested, will pass the requirements and the specifications of the American Society for Testing Materials, and in America they supplement this by propaganda concerning the manner in which cement should be used, the kind of water, sand and gravel that is to be used with it, the weather conditions under which it shall be placed, all of which are commendable efforts. The protection of their own interest, however, requires that they go one step further and make it their business to see that the conditions under which

cement gets into a completed structure are such as will be a credit rather than a liability to the concrete industry.

Under the social system in Japan up to the present time, the higher class Japanese has received technical training, and it is rather detrimental to the standing of the higher class to come into close contact with real work and dirt. As a result, designs and specifications which may have been properly prepared are not checked by the same individual in person to see that the design and specification is actually incorporated into the work. It is true inspections are made, but they lack the intimate contact which knows that the thing is being done as it was intended to be done.

Another point in connection with the social system is that the government officials are in position of supremacy. They are in control of the issuance of permits and do inspect plans and specifications. (The average Japanese takes his hat off before speaking to a policeman.)

They fail, however, to closely examine the actual construction, for the same reasons as stated before.

This same social system, while it causes an owner to feel in honor bound to prevent a contractor from suffering any actual loss in connection with the performance of a contract, also requires the contractor to set himself up in a pretty favorable position in order to get consideration in case he has bid low for the work.

The Japanese as an individual is extremely conservative where his purse is concerned. His continual endeavor is to find some way to construct cheaper than others are or may have been doing. This desire has a tendency to develop skimped designs, particularly among those who fail to appreciate the possibility of disaster by such action.

Construction work, for the most part, is awarded on a lump sum basis, after the severest kind of competition. The work is awarded to the low bidder in many cases, and frequently at a price at which the work cannot be produced. The contractor immediately endeavors to find some means of reducing the cost to permit him to come out whole, and frequently does something that is not safe.

Anyone who is certain he can build real earthquake proof structures can make a lot of money by placing himself in position to make his guarantees safe ones for the Japanese to accept. He will thus have no difficulty in obtaining plenty of business when contracts are supported by such guarantees.

General.—The possibility of a repetition of a "quake" as violent as that of September 1 is remote. That it will occur soon is equally remote. There is no assurance, however, that the next one will not be much more violent than this one. The question, therefore, arises, can we build an entirely safe building; how much will such building cost?

In my opinion a structure reasonably safe from collapse can be constructed at very little increase in cost, except for foundations. It is doubtful if side walls and partitions using existing materials can always

be expected to come through unharmed, except considerable expense is used in their construction, and then they will be questionable.

Height of building, at least up to eight stories, apparently is little more of a problem, if any, than on heights of two and three stories. Reliable men who have experience in designing and in constructing them are more limited.

In my opinion a foundation safe within reasonable conditions (bearing in mind September 1) can be constructed of one of the several types in use today, on most any kind of bearing soil, except it be very soft to large depth.

If, however, you have a site to choose, do not select one where different types of foundations will be required, and by all means avoid one where fill (or wash) has been laid against a steep, hard soil or rock.

Buildings should be designed to have their weight well balanced all around the center point. Floor area adds to the safety of the superstructure, but detracts from that of the foundation above and below a certain point. In general, the height should not exceed the width. Length, after you have enough to brace the superstructure, should be limited in monolithic and multiple story building. Avoid heavy ornamentation and all possible weight, especially near the top. Be certain you have structural strength in all directions and good work performed for you.

Their methods of approaching business are at variance with ours. They serve tea and inquire about all of your family as a means of putting everybody at ease before undertaking the problem to be discussed and on important matters may send you a valuable present. They are extremely courteous, and, in general, could give us education along this line.

Their people possess the same feelings in regard to business integrity, pride in accomplishment, good standing among their fellow men, etc., that we do. They depend upon these things in business, because their laws are less forceful in holding one to the letter of the law of contracts.

Conclusion.—I feel my remarks would be incomplete without making some statement in regard to Japanese business methods. There are classes of people in Japan just as there are in America. Some follow sharp practices in their business methods. We were advised to be very careful of of ourselves in undertaking business arrangements in Japan. We have endeavored to be careful, but have found that when a Japanese business man of standing in Japan has a complete and thorough understanding with you, he has the same feeling of responsibility in connection with it that our business men of the same kind have.

BUILDING CODES AND THEIR RELATION TO THE CONCRETE PRODUCTS INDUSTRY.

By William F. Lockhardt.*

Building codes today are among the outstanding problems of the concrete products industry.

The average building code as at present written fails to prevent the manufacture of poor units; does not protect the manufacturer of quality products, and falls short of allowing the prospective user of concrete products the freedom necessary to enable him to make the most of their possibilities for economical and substantial construction.

Few building codes take cognizance of the enormous strides that have been made in the technology and mechanics of concrete products manufacture in the last decade. There is no recognition of the improved quality of unit which has resulted therefrom. Codes are still written in such a way that the manufacture of high-quality products is not compulsory, and as a natural result onerous restrictions are imposed on their use.

It cannot be denied that poor quality concrete block have been made in the past. It should be very clearly understood, however, that under no conditions need anything but the very highest grade of block be made today. Where the contrary condition exists it is directly the fault of a weak or antiquated building ordinance, or the absence of any ordinance.

Primarily, the reason for poor products lay in the fact that theoretically, at least, concrete block could be made by any man with sufficient capital to buy a flimsy hand machine from a mail-order house. For entry into concrete products manufacture, prerequisites in the shape of capital and technical and business ability have been unnecessary.

As a logical consequence, block were made in such small quantities that in a pinch the only practicable economy was that of still further reducing the cement content. Uncertain hand methods of manufacture added a further element of doubt. That enough water could be used in the mix to properly hydrate the cement and curing after manufacture to overcome this deficiency was unthought of.

The unit itself, pasty-faced and dull, was molded into panel or imitation rock-faced form which precluded its use for anything but exposed work and made it anathema to architect and discriminating builder alike.

Today the concrete structural unit is being offered for the straight utilitarian purposes for which it is best adapted, namely, for load-bearing and back-up work, where common brick are used, and as the best possible

* Portland Cement Association, New York City.

base for portland cement stucco. It no longer attempts to masquerade as something that it is not.

The advent of high-production machinery and a lighter and smaller type of unit have both combined to bring into the field men of capital who have both the business and technical ability to conduct their operations on the highest plane. They are compelled, if for no other reason than to safeguard their heavy investment, to make only a reputable, high-grade product of consistent excellence. Their manufacturing operations are designed to give the best results; they are making use of the latest data available on the effect of the grading of their aggregates; on the influence of proportions, amount of mixing water used, time of mixing, length and time of curing, and so on indefinitely.

The result is that the concrete building unit made in a modern plant is almost unique among structural materials in that its quality is completely under the control of the manufacturer. The older and long accepted masonry units cannot approach the properly made concrete block, brick or tile in this respect. A moment's thought will show that under the conditions mentioned above the concrete unit is made by combining a definite amount of portland cement and a predetermined quantity of aggregates. A fixed amount of mixing water is added and the batch mixed for a certain length of time in a machine operating at a fixed speed. The manufacturing machinery is semi- or full-automatic, subjecting the concrete to a definite number of tamping or compressing blows, after which it is cured for 24 to 36 hours in moist steam or under a water spray at a fixed temperature. Possibly not all plants combine all of these methods, but they are so closely followed that a constant quality can be maintained commercially.

But building codes, by and large, are still predicated upon the assumption that only the hand-made product is in the market. They allow the Building Inspector little if any latitude in interpretation.

The Building Inspector in the average small or medium-sized city is almost without exception earnestly conscientious in his application of the Code under which he works. As a rule, however, he is not a trained engineer. Most frequently he is a former carpenter or mason contractor of the old school who looks upon the concrete building unit with misgiving, having in mind the many dubious products he has seen. Unless a modern plant happens to be located in his immediate vicinity it is quite probable that he has no idea of the extent to which the industry has progressed, and not the faintest conception of the fact that good concrete products are no longer manufactured by rule-of-thumb, but are the result of a closely controlled manufacturing operation comparable to that required for the manufacture of other standardized articles.

Without this knowledge on his part the community and industry suffer. The code as written gives him but limited power at best in dealing

with concrete block; frequently concrete block are not mentioned at all and therefore cannot be allowed. Perhaps he would like to have tests made on suspicious cases, but where are funds to come from when his own office is frequently only a part time one and his stipend held down to perhaps $500 per annum?

If there develops in his own town sufficient demand for block, or if a plant is started nearby under aggressive management, however, then comes a time when one of two things happens: In trying to sell some big job the reputable manufacturer either runs afoul of some antiquated ruling in the Code and finds his product barred, or he finds this job taken away from him by one or more fly-by-night operators who can undersell him by making an inferior product.

Both cases are serious. The first is a curtailment of what should be a legitimate outlet; the second is, if anything, worse. Allowed to continue unrestricted, the manufacture and sale of inferior block will arouse or maintain such a prejudice against the concrete unit that not only will the market be curtailed—it will automatically be eliminated altogether, the good and bad suffering alike.

A movement for building code revision is then started. If successful, a committee is appointed consisting of the Building Inspector and three or four local business men, and after the usual preliminaries the actual work of preparing the new code falls to the Building Inspector. When the draft is completed it is usually submitted to one or two or three disinterested technical organizations for criticism and suggestion. It is checked for compliance with standard building code features,—allowable stresses and loads, fire protection, public safety, etc., and, if satisfactory, returned with such comment as may be necessary.

Concrete building units are generally passed by with the same mental picture as that held by the Inspector and as long as two or three time-honored limitations on their use are included they are given no further thought. In a few cases self-styled "Building Code Experts" are retained to write a new code, which is not infrequently even less satisfactory all around. They have less opportunity than the Building Inspector for finding out just what progress has been made in that locality in products manufacture, so that if concrete block are mentioned at all, they are disposed of by copying verbatim pertinent paragraphs from some older and less progressive ordinance than is required to meet modern conditions.

For these reasons it is important that the products manufacturer maintain the closest possible contact with the work of code revision. Regardless of how favorable at the start some of the members of the committee may seem to be, they cannot forget that he is an interested party, presumably biased in favor of his own product, and that in the last analysis they are not technical men, capable of passing upon the fine points that he raises. They are more than likely to subordinate their views to the rec-

ommendations of outside authorities unless an intimate contact can be maintained to answer questions and present evidence while their opinions are still in a formative stage. After the tentative draft has been published it is too late to do this work effectively. When a man has finally committed himself publicly to a certain course of action he is naturally bound to defend it, so that if he is contrary minded his point of view must be changed, if possible, before he has had a chance to commit himself by allowing his name to be published as one of the sponsors of the new legislation.

Because most building codes are modelled along fairly definite lines —if not at first, at least after they have been through a process of criticism and review by some authority—it is usual to find in them certain restrictive clauses that are founded on erroneous assumptions, omission of necessary sections relating to the control of quality, etc., with regard to which the manufacturer and Building Inspector should have the fullest information. The review of these sections and the setting forth of such data as is available is the purpose of the second section of this paper.

PART II.

Probably the most satisfactory method of handling concrete building block and tile in a building code is to devote a section of the code to them alone. This method was followed in Camden, N. J., in an ordinance adopted Sept. 28, 1922, and in Schenectady, N. Y., in July, 1923, for example.

Such a section can contain all clauses relating to the manufacture, sale and use of block, greatly facilitating reference and use. To assist in getting it adopted where it is found possible to get new regulations relating to concrete products it is recommended that it be self-contained, thus rendering it independent of other sections of the code.

The concrete products section may then be sub-divided somewhat as follows:

1. Introduction
2. Materials
3. Proportions
4. Tests Required
5. Methods for Making Tests
6. Standards of Quality
7. Use
8. Working Stresses
9. Thickness and Height of Walls
10. Party Walls
11. Construction
12. Licensing
13. Identification
14. Penalty

An ordinance adopted and actually in force in two cities has passed the stage of theory. The code adopted by Camden, N. J., was accepted

only after a very careful study of all factors relating to the manufacture and use of block, including the necessity of curbing manufacturers who had been making inferior block, or shipping what would have been good block, at too early an age. As there was a definite reason back of every paragraph in this code which would apply with equal force in other communities it will be reviewed briefly by sections.

AN ORDINANCE REGULATING THE MANUFACTURE AND SALE OF CONCRETE BLOCK AND CONCRETE STRUCTURAL TILE FOR BEARING WALLS AND PIERS IN CAMDEN, N. J.

(Adopted Sept. 28, 1922.)

Section 1. Be it ordained by the City Council of the City of Camden, N. J., that building units, such as concrete block and structural tile which shall be sold for use in bearing walls and piers erected in the City of Camden, N. J., shall be made from portland cement and suitable aggregate and shall be classed as concrete block and shall comply with the requirements of this ordinance, hereafter stated.

MATERIALS.

Section 2.

(a) Portland cement shall conform to the standard specifications of the American Society for Testing Materials,* (1315 Spruce Street, Philadelphia).

(b) All fine and coarse aggregate shall be clean and free from deleterious substances.

(c) Fine aggregate must consist of sand, gravel, crushed stone, crushed blast furnace slag, steam boiler cinders or other equivalent material suitably graded from fine to coarse and which will pass, when dry, through a screen having openings one-fourth of an inch in diameter.

(d) Coarse aggregate must consist of gravel, crushed stone, crushed blast furnace slag, steam boiler cinders or other equivalent material which, when dry, is retained on a screen having openings one-fourth of an inch in diameter and which will pass through a screen having openings three-fourths of an inch in diameter.

The most important features of these sections is that they give a very comprehensive definition for fine and coarse aggregates, comprising all the commonly used materials. Crushed blast furnace slag has been made the subject of exhaustive tests by the Bureau of Standards at Washington and found to be entirely satisfactory, the strengths running about the same as for crushed stone in most cases.

*Standard Specifications and Tests for Portland Cement, adopted Jan. 1, 1917, with amendments effective Jan. 1, 1921.

Cinder block have been in use for over twenty years in some parts of the country and have been extensively used during the last seven years in the cities of Philadelphia, New York, Buffalo, Rochester, N. Y., Albany, Camden, N. J., New Kensington, Pa., etc. They have been tested and approved by the Underwriters Laboratories, Inc., with a rating of 2½ hr. as a fire retardant, for an 8-in. block wall, unplastered.

PROPORTIONS.

Section 3.

(a) Materials for concrete block shall be so proportioned that the block shall meet the requirements of the compression and absorption tests hereinafter specified.

(b) Hydrated lime may be added to the mixture not to exceed 10 per cent of the volume of the cement used.

It is obvious that the efficiency of manufacturing methods has a decided bearing upon the amount of cement needed in any mix to attain a given strength. Higher pressure or heavier tamping will give a stronger block than can be obtained under inefficient hand methods with a richer mix. Grading of aggregate, length of mixing, control of mixing water, etc., all affect the strength of concrete aside from the quantity of cement used. If acceptance is conditional upon obtaining a certain strength, the manufacturer should be left free to get that strength by any mix that he can maintain consistently.

TESTS RECORDS.

Section 4.

(a) Concrete block shall be subjected to the following tests before their use is approved by the Building Department.

(b) *Compression*—All concrete block intended for use in bearing walls and piers, shall be subjected to a compression test.

(c) *Absorption*—Concrete block shall be subjected to an absorption test.

METHODS FOR MAKING TESTS.

Section 5.

(a) For these tests at least three samples shall be selected at random from manufacturer's stock or from block delivered to job, by a representative of the Building Department. The samples shall be of the regular size as used in construction and must be marked for identification. In case of failure of first three specimens to meet requirements, three more specimens shall be taken and the test repeated. The second test shall be final.

(b) These tests shall be made in a testing laboratory of recognized standing, in accordance with standard methods prescribed by American Concrete Institute.* All tests shall be made at the expense of the manufacturer, dealer or selling agency.

---

*Standard Specifications for Concrete Building Block and Concrete Building Tile, American Concrete Institute, 1807 East Grand Blvd., Detroit, Michigan.

(c) The Building Inspector shall require, at least every four months, a certificate of test of product made by every manufacturer and selected by a representative of the Building Department. The Building Inspector may require additional tests to be made on samples selected by him whenever, in his opinion, there is doubt as to whether the product meets the standards prescribed by these regulations.

Paragraph (a) provides reasonable assurance against the manufacture of special block for test purposes by providing that block may be taken from stock delivered to job.

By specifying that all tests shall be made at the expense of the manufacturer, etc., the Building Inspector is relieved of the necessity of including in his budget each year an appropriation for the testing of block. He is under this provision left free to make tests at least three times per year and oftener if conditions warrant. The cost of testing need not be burdensome to the manufacturer, as most schools and laboratories having the necessary testing equipment are more than glad to co-operate in any work that will raise the standard of quality in construction work. In several Eastern cities for example the local colleges have made a special flat rate of $5.00 to cover a series of five compression and two absorption tests, when made by manufacturers or manufacturers' associations for the purpose of improving quality, subject to the condition that all block to be tested be delivered at one time, so that the tests could be made with one set-up of the machine.

STANDARD OF QUALITY.

Section 6.

(a) The ultimate compressive strength of hollow and two-piece block at 28 days or when delivered to site, as determined by three test specimens, must average 750 pounds per square inch on the gross cross sectional area of the block as designed to be used in wall and must not fall below 650 pounds per square inch in any test.

(b) The ultimate compressive strength of three test samples of solid concrete block shall be not less than 1200 pounds per square inch of gross cross sectional area.

(c) The absorption test will be made upon the specimens before they are used for the compression test.

(d) The absorption of three test samples when dried to a constant weight at a temperature between 225 and 250 degrees Fahrenheit and placed in clean water with the top surface exposed to the air for a period of 48 hours, shall not exceed 12½ pounds per cubic foot of concrete (actual volume) contained in any block.

While the A. C. I. standards classify block for four purposes with four corresponding strengths such a separation is out of the question in

the average building code for small or medium-sized cities. The Building Inspector will in all likelihood say that he has enough trouble (if he attempts to enforce a quality standard code) without adding the complication of three additional permissible grades. Such an arrangement is undoubtedly desirable in large cities where it is desirable to allow quality manufacturers the widest scope for the sale of high grade units, but in the small town one grade is all that can be practicably maintained.

The N. Y. City and the N. B. F. U. Codes call for 750 lb. per sq. in. gross area, while other codes go 50 lb. lower or higher, but the general average is in the neighborhood of 750 lb., with a safety factor of 10 giving a working stress of 75 lb. per sq. in. gross area.

Following the N. Y. and many other codes the strength provision generally reads somewhat as follows: "Compressive strength per sq. in. gross area shall be not less than 750 lb. when tested with the cells vertical, and 300 lb. when tested with the cells horizontal."

This two-fold test arises largely from the fact that in many localities hollow clay units are made with a 12 x 12 in. face as seen after being laid in the wall, so that they may with equal facility be laid in two positions. Their strength is greatest when laid with the cells vertical, but on account of the narrow mortar bed thus afforded, the average mechanic will prefer to lay them on their side at a sacrifice of strength in the wall. Concrete units have usually an 8 x 16 in. or a 5 x 12 in. face and therefore are rarely if ever laid any other way than as intended—with the cells vertical. For such special types of unit as are intended to be laid with the cells horizontal there is no reason for reducing the strength requirement. Such paragraphs should therefore read as given in the Camden Code, Section 6, of this report.

A two-story masonry dwelling will impose loads averaging about 25 lb. per sq. in. on the foundation course. A four-story warehouse 40 ft. high, uniformly loaded over a 20 ft. span to 125 lb. per sq. in. on all floors, will not attain the allowable 75 lb. per sq. in. on the flooring course, even if an 8 in. wall were used. The concrete block wall therefore has ample strength to carry any loads that might be imposed upon it within the limits of height for which it can be used.

This strength requirement is the only logical basis upon which to admit hollow structural units. Attempts to achieve the same end by the more complicated means of restricting the web thickness, specifying the mix or the strength per sq. in. of net section serve no purpose other than to make it difficult to establish a standard of comparison between different units and different codes.

Absorption.—The maximum allowable absorption for concrete building units usually coincides, approximately at least, with the minimum for clay products. Medium hard clay brick under the specification of the American Society for Testing Materials may absorb between 12 and 20 per cent and not to exceed 24 per cent for example.

The purpose of leaving the upper face of the block or tile exposed when making the absorption test is to permit the escape of the air in

concrete, which would otherwise be held in the concrete were the block completely submerged.

The criticism is rightly made that this immersion test is not comparable to any requirement in actual practice. It has been suggested that the unit to be tested be set in a pan containing ½ in. of water, instead of being almost completely immersed. Its capacity for absorption, or capillarity, will then be shown by the extent to which the water in which it is set is drawn up into the unit.

When the block is immersed, the test really becomes one of water carrying capacity rather than actual absorption, such as would draw moisture through a wall. Cinder block, with relatively large pores, will often carry more water in such a test than will a sand block, yet because the large pores reduce capillary action under normal exposure in the wall, there is often less likelihood of moisture being drawn through the block to the inner face of the wall. In many places where cinder block have been used for years without protective covering of any kind the walls remain perfectly dry on the inner face during prolonged rains.

It should be further borne in mind in setting absorption limits that 10 per cent allowable absorption based on the weight of the block is approximately 5 lb. of water in the case of sand block, while a cinder block of exactly the same size and type, because of its lower weight (30 lb. instead of 50 lb.), would be allowed only 3 lb. absorption. This is manifestly unfair to the cinder unit.

The method adopted in the Camden code avoids having to specify different absorption percentages for different aggregates. Sand concrete used for the manufacture of concrete building units will weigh about 125 lb. per cu. ft., on which basis 10 per cent absorption would be 12½ lb. The allowable absorption is therefore set in pounds of water per cubic foot of concrete contained in the block, regardless of the weight of the block or the aggregate used.

In the proposed standard specifications of the American Concrete Institute for concrete building block and tile a formula is given to correct for this variation in weight as follows:

"Concrete block and tile shall not absorb more than ten per cent of the dry weight of the unit when tested as hereinafter specified, except when it is made of concrete weighing less than 140 lb. per cu. ft. For block or tile made with concrete weighing less than 140 lb. per cu. ft., the absorption in per cent by weight shall not be more than 10 multiplied by 140 and divided by the unit weight per cubic foot of the concrete under consideration."

USE.

Section 7.

Concrete building block which have the approval of the Building Department may be used for buildings wherever solid masonry is permitted by this Code, subject to all the limitations required for solid masonry and subject to the allowable working stresses and provisions of this Code.

WORKING STRESSES.

Section 8.

(a) The maximum allowable working stress for masonry walls, piers and pilasters of hollow concrete block or tile shall not exceed 1/10 of the average crushing strength of the block when laid in lime-cement mortar and shall not exceed ⅛ of the average crushing strength of the block when laid in portland cement mortar.

(b) The maximum allowable working stress for solid walls, piers and pilasters of solid concrete block shall not exceed ⅙ of the average crushing strength of the block when laid in portland cement mortar.

(c) Where the height of piers between lateral supports exceeds five times the least lateral dimension, the maximum allowable working stress shall be the reduced unit stress obtained by the use of this formula: The quotient obtained by dividing the height of the pier in feet by twenty, times the least horizontal dimension in feet, shall be subtracted from 1.25 and the remainder multiplied by the maximum allowable working stress per square inch as given in paragraph (a) of this section.

THICKNESS AND HEIGHT OF WALLS.

Section 9.

(a) For residences or dwelling houses, the following table shall determine the minimum thickness of superstructure of concrete hollow block or tile:

No. of Stories	Basement	1st Story	2nd Story	3rd Story
1	12″	8″		
2	12″	8″	8″	
3	16″	12″	8″	8″

(b) In no case, however, shall the total height of any bearing wall of hollow concrete block or tile in any building exceed 40 feet.

Allowable Height of Structure.—Codes are often written to read: 3. Limitation. "Walls of hollow building blocks shall not be used in buildings over forty feet in height, etc."

This in many instances has worked a hardship to all concerned where the height of the building was just over 40 ft. This particular requirement is one that probably cannot be completely overturned at the present time, and yet is one that should be interpreted rather liberally, in the light of the reason for this limitation.

The height of a wall built of hollow units is restricted not because of any lack of strength to carry direct leads but because of two things: (a) The fact that most hollow units subjected to a fire on one side will suffer a shear through the transverse or connecting webs immediately back of the heated face, by reason of unequal expansion, and (b) The lighter

weight of hollow walls which implies a decreased resistance to overturning as a result of the lateral thrust of falling beams in a fire.

With regard to (a) Cinder and limestone aggregates are less affected by exposure to fire than quartz concretes and hollow clay units. Certain cinder concrete block, it is interesting to note, have been passed by the Underwriters Laboratories, Inc., and seem to be remarkably free from this shearing action, coming through the standard Underwriters fire tests with strength very little impaired. Other intensive tests on blocks made of various aggregates have been conducted by the Underwriters Laboratories, and a report will be forthcoming shortly. How this shear through the transverse webs of a block or tile affects the ability of a wall to carry its load has not as yet been made the subject of investigation, but because of the splendid bond developed between concrete building units and the cement mortar in which they are laid, it should be higher than for other hollow wall units.

Concerning (b), this view of the underwriter is based on the results of a study of the attitude of fire chiefs the country over and must be accepted, even though the expression of opinion is by no means unanimous. Where a fire resisting floor construction is used, however, this argument loses considerable force.

Many codes accept hollow walls for four story buildings and it would seem desirable, where possible, to make the *number of stories* the basis for limitation, rather than the actual height in feet. It is generally agreed that though it may seem desirable to limit the height of the hollow block section of a wall, there should be no objection to allowing the builder to use hollow block or tile construction if the building happens to be between forty and say, forty-five feet in height, just as long as he does not run over four stories. As an alternate he may be allowed to use up to say five feet of brickwork on top of the foundation in order that the hollow section of the wall may not exceed the prescribed 40 ft. limit.

Such a paragraph might be worded as follows: "Hollow building blocks of concrete may be used for all walls of buildings not exceeding four stories or 45 ft. in height, provided, however, that the hollow concrete block section of such walls shall not exceed 40 ft. in height. The minimum thickness of such walls shall be as required for brick walls."

A table of thicknesses for hollow block walls for four-story buildings (dwellings) would run about as follows:

Foundation	16 in.
1st Story	16 "
2nd Story	12 "
3rd Story	8 "
4th Story	8 "

This follows the same arrangement as the Camden code. With the four floors framing into the wall in less than forty-five feet of height the wall should be quite rigid.

PARTY WALLS.

Section 10.

(a) The minimum thickness of party or division walls of hollow building tile or hollow concrete block or of hollow wall construction shall be 8 in. for spans not exceeding 18 feet and 12 inches for spans not exceeding 24 ft.

(b) Hollow building tile or hollow concrete block in a party or division wall may be broken into for the insertion of building members; such members, however, must not butt, but must be staggered.

Concrete block make very economical and satisfactory division and party walls for row construction, 12 in. hollow block being accepted for this purpose in the Department of Commerce Building Code. It is recommended that the bearing course be filled solidly, as well as the space between the beams, and the course above. Joists framing into opposite sides of party walls should be separated by at least 4 in. of masonry.

There is a point not covered in the Camden code that is frequently found under the sections relating to reinforced-concrete frame and steel frame construction. We refer to curtain, or panel, walls in such buildings.

In the larger cities where a skeleton form of construction is used with the curtain or panel walls supported on beams at each floor, terra cotta block are commonly accepted for backup, but no provision is made to accept concrete units for this purpose. In new codes the wording should be such that concrete building units are admitted wherever clay products are. The only reason they have not been so used in the past has been the limited output with resulting high cost. Where a sufficient volume of output has been attained they have been immediately used for all types of construction, as in Detroit, where they are regularly used for backing up brick veneer in skyscraper work.

Panel walls are frequently required to be 12 in. thick. Such walls only serve to keep out the weather, however, and in some cases 60 per cent of the area between columns is nothing more substantial than glass. Where 8 in. construction is accepted for bearing walls for two story dwellings there can be no valid objection to the use of 8 in. non-bearing panel walls. Furthermore, the second Hoover Committee report, issued in tentative form on Sept. 1, 1923, under the title of "Recommended Minimum Requirements for Masonry Wall Construction," specifically recommends 8-inch masonry panel walls of material such as hollow concrete block, brick, clay tile, etc.

CONSTRUCTION.

Section 11.

(a) All concrete block shall be laid in portland cement mortar as defined in this ordinance with vertical joints broken, and with all courses thoroughly bonded. All masonry facing of concrete blocks shall be bonded to the backing as required for the kind of facing used.

(b) All masonry backing of hollow or solid concrete blocks shall be bonded to the facing as required for the kind of facing used.

(c) Whenever concrete block is to be covered with stucco or plaster, the surfaces to receive same shall be roughened, or joints struck off roughly and not pointed.

(d) All bearings on hollow or solid concrete block construction shall be not less than four inches. Where vertical cell construction is used the load shall be distributed by means of metal or masonry bearing plates of sufficient thickness to distribute the imposed load or the supporting course filled with concrete; or other equivalent method of construction may be used.

(e) Wherever a change occurs in the thickness of walls or piers of hollow concrete blocks laid with the cells vertical, unless the webs and shells are properly superimposed, the bearing loads shall be distributed upon the wall below by means of metal or masonry bearing plates or the supporting course be made of solid block, or other equivalent method of construction may be used.

In dealing with the subject of "Construction" there are several practices commonly required that are wasteful because they add to the cost of construction without yielding any commensurate return to the owner.

One of these is the requirement relating to the filling of hollow concrete block foundations. Many codes contain a provision that hollow concrete building block must be filled with concrete when used for foundations. This is chiefly with the idea that the added weight is necessary to resist earth pressure. Hollow block foundations are accepted in many cities, where they invariably stand up and give good satisfaction. In some cases the thought is to assist in making a watertight wall, but it seems to be a poorly chosen means to that end. In dry sandy soil, such provision is not needed if the block are properly laid. In wet soils or where there is ground water, filling the block is not enough to compensate for possible deficiencies of workmanship. The usual practice in trenching and laying drain tile should be followed and a plaster coat of cement mortar applied to the outer face of the wall, with a paint coat of some asphaltic or bituminous compound applied before backfilling, if necessary, the same as with any other type of masonry below grade.

It is common practice to require 12 in. walls to grade. This also appears to have its roots in the questions of strength and water resistance mentioned above and to be subject to the same criticisms. Except for cellars of unusual depth and length between cross walls, say over six feet below grade, and more than forty feet between cross walls or buttresses, 8 in. block have given satisfaction.

Unintelligent restrictions calling for uniformly heavier construction than is required for the average case only add an unnecessary burden on the construction industry. Special cases should be recognized and handled as such.

LICENSING.

Section 12.

Every manufacturer of such tile or concrete block, selling agency or dealer therein must secure license for the manufacture or sale for use for building purposes for such block or tile in the City of Camden, N. J., for which license there shall be paid to the Building Inspector for the use of the City, a license fee at the rate of $50 per annum or for any part of any calendar year. All such licenses shall expire on December 31st of each year. Said license shall be granted as soon as certificate of test has been obtained and approved by the Building Inspector showing that the concrete block proposed to be sold have met the requirements of this ordinance. The name of the firm or corporation and its responsible officers making application for such license must be placed on file with the Building Inspector before any such license shall be issued. All changes in ownership or management of any license must be reported in writing within 5 days thereafter to the Building Inspector. In case of such change any such license may be transferred by the Inspector on the payment of a transfer fee of $3.

IDENTIFICATION.

Section 13.

A mark of identification shall be impressed on every block to be manufactured, sold or used in the City of Camden by the maker or dealer thereof, before said block leaves the yard or salesroom of the maker or dealer. A copy of the mark shall be filed with and approved by the Building Inspector prior to the issue of any license hereinunder.

PENALTY.

Section 14.

(a) Any person, firm or corporation violating the terms of this ordinance by manufacturing or selling or using concrete block which do not carry an identification mark, or which have not been licensed shall pay a fine of $25 and the costs of prosecution for each offense. In case such person, firm or corporation shall violate the terms of this section of this ordinance, the Building Inspector shall forthwith give notice to such offender to remove such block and each day and any part of every day such block remain, after the service of any such notice shall be taken and considered and construed to be a separate and distinct violation of this ordinance for which such offender violating same shall pay a fine of $25 and the costs of prosecution.

(b) Manufacturer will be held responsible for seeing that block are not delivered to the job until they are 28 days old and have attained the strength specified hereinabove.

(c) If samples of block taken after license has been granted

fail to meet the compression and absorption tests as hereinbefore set forth, the penalty for failure to meet the requirements of this Code shall be, for the first failure, a fine of $100; for the second failure, a fine of $200; and for the third failure, suspension of license to manufacture or sell in the City of Camden for a period of one calendar year from the date of such failure.

(d) In addition to the penalties above provided, in case any licensee hereunder shall violate any of the terms of this ordinance, any such violation shall be good cause for the revocation of such license. Any such license may be revoked for any such violation after notice and on hearing before the Building Commission.

The Building Inspector, in any such case, shall prefer and sign the charges and give the necessary notice whenever in his judgment any such license should be revoked for the violation of any of the terms of this ordinance. It shall be the duty of the Building Inspector and of his assistants, from time to time, to inspect all concrete block manufactured or used in the City of Camden, and to report monthly to the Building Commission any violation of this ordinance.

NOTE ON CAMDEN BLOCK AND TILE ORDINANCE.—Paragraph (a) in Section 11 of the above ordinance states that all block must be laid in portland cement mortar. Paragraph (a) of Section 8, however, specifies a working stress for block laid in lime-cement mortar. Where it is desired to permit the use of portland cement mortar only, Section 8 should be changed accordingly. If lime-cement mortar is to be permitted, Section 11 should be changed, and lime-cement mortar should be defined as mortar containing one part of portland cement and one part of lime to not more than six parts of fine aggregate, all proportions to be by volume.

In view of the fact that manufacturers of other building material are not required to be licensed there may be complaint at times from the products manufacturer that his industry is being discriminated against. It may be that from some standpoints the practice is indefensible. On the other hand it must be admitted that it is possible for the fly-by-night operator to make a poor product unless there exists the power to curb him, while under such an arrangement the reputable manufacturer gains by being able to advertise that his is the only mason material manufactured to conform to standard specifications under building department supervision.

The periodical tests are made at his expense and the reports are therefore his property. They should be the basis of his selling effort. A license fee will not bankrupt him, while it will serve to deter the adventurer, as does the requirement for testing, if properly administered. Finally, to be effective, such an ordinance must have teeth, and sharp teeth at that.

While there may be a division of opinion as to how far a code can rightfully go in the matters of licensing fees, test requirements and penalties, the Camden code represents a carefully thought out effort to make a worth-while structural unit available for the reduction of building costs on a basis which would allow the honest manufacturer the utmost freedom from petty restrictions, while at the same time retaining the power to check instantly any manufacturer who might attempt to market an inferior product.

There is nothing inherently mystifying about the process of getting good concrete building units. Nor can it be assumed for a moment that the majority of products manufacturers need watching. But it is entirely possible to make a poor product if a manufacturer so wills it, and to prevent this the reputable manufacturer can well afford to take the position of supporting a strictly quality code.

It cannot hurt him, and should help him, and if by ensuring that only products of the highest quality can be made he can get them approved in return for a wider range of uses his market is immeasurably broadened and the industry as a whole correspondingly helped.

The Relation Between the Wall Thickness and Diameter of Concrete Culvert Pipe for Highway Use.

By G. W. Hutchinson.*

The manufacture of concrete culvert pipe appears to be an easy task. On the contrary, it is a matter which requires considerable study if an economic product is to be secured and also is one in which continual improvement is possible.

The control of detail in making concrete culvert pipe is more exacting than that ordinarily exercised in large concrete structures such as buildings, bridges or highways, but cannot be expected to comply with the demands of laboratory procedure. The practice in general, however, should lean toward the laboratory control basis, as concrete placed under field conditions varies too much in quality to allow culvert pipe made from it to be within a fair margin of uniformity. There are several methods of producing a culvert pipe having a high crushing strength, but the manufacturer who does this economically must consider and take advantage of certain parts of all methods. Such methods do not dwell entirely upon the purchasing of so-called first-class commercial materials, but cover every detail from the intelligent selection of each material, the consistency of the mixture, the time of mixing, to the placing and curing of the concrete.

Assuming that the method used in manufacture is one by which a uniform concrete is secured, it is necessary to have information regarding the necessary wall thicknesses for culvert pipe of different diameters in order that the ultimate crushing strength of the pipe will be proportional to the load it is required to carry in the field. With a uniform concrete, the wall thickness is the governing factor in producing an assortment of different diameters of pipe to meet a specification requiring a crushing strength proportional to the diameter of the pipe, usually expressed in pounds per linear foot multiplied by the inside diameter of the pipe in feet.

A typical curve showing the crushing strength of different sizes of culvert pipe having the wall thickness usually required for highway work is shown in Fig. 2. This represents an average of tests made on concrete pipe from ten different pipe manufacturers.

---

* Formerly Assistant Engineer, North Carolina State Highway Commission.

An example of the difference in crushing strength of the specimens due to variation in the materials and manufacture is given as follows:

Diameter of Pipe, in.	Mean Variation from Average, per cent.
15	19
18	24
24	28
30	25

The result of these tests indicated so much variation in crushing strength, either above or below the usual specifications, that a laboratory investigation was carried on to determine how much of this variation was due to manufacture and to determine if the requirement for wall thicknesses specified for the different diameters was consistent with that which would be necessary to produce a given crushing strength on the foot-diameter basis.

The series included in this paper deals entirely with the relation of wall thickness to the diameter of the pipe. No work on the value of reinforcement is included and the results given are for pipe with plain concrete walls.

The investigation consisted of tests of about ninety pieces of culvert pipe. They were cast in special molds conforming to the diameter and wall thickness given and were made in one foot sections. The results in general are the average of three or four specimens (Table I). The result of the tests of the 36 in. pipe with a 5 in. wall thickness will be found to be apparently erratic. This is probably due to there being but two specimens for an average, and it will be noted in Fig. 5 that the value on the curve including this result is taken as an average between the actual result and the one obtained by prolonging the curve in Fig. 4.

Ten specimens were made each day. The investigation was laid out so that the results of one day's work would interlock (see Fig. 1) with the rest of the series. The original program called for one hundred pieces or ten days' mixing, but on account of a shortage of cement, set aside for this investigation, it was only possible to make specimens from nine days' mixing. Several pieces were fractured while being removed from the molds which allowed only an average of two or three pieces in these cases.

The concrete was mixed in a machine mixer and a batch of the same size was used for all specimens so as to eliminate any variation in the quality of the concrete due to changes in the size of the batch.

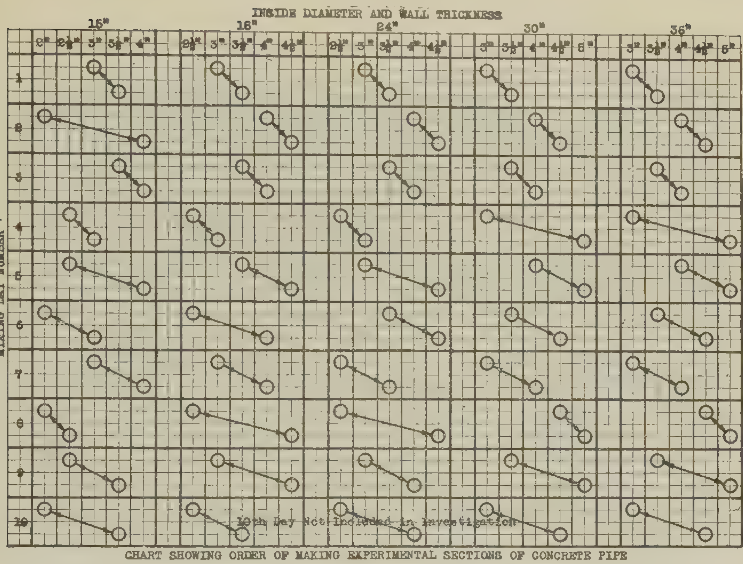


FIG. 1.—CHART SHOWING ORDER OF MAKING EXPERIMENTAL SECTIONS OF CONCRETE PIPE.

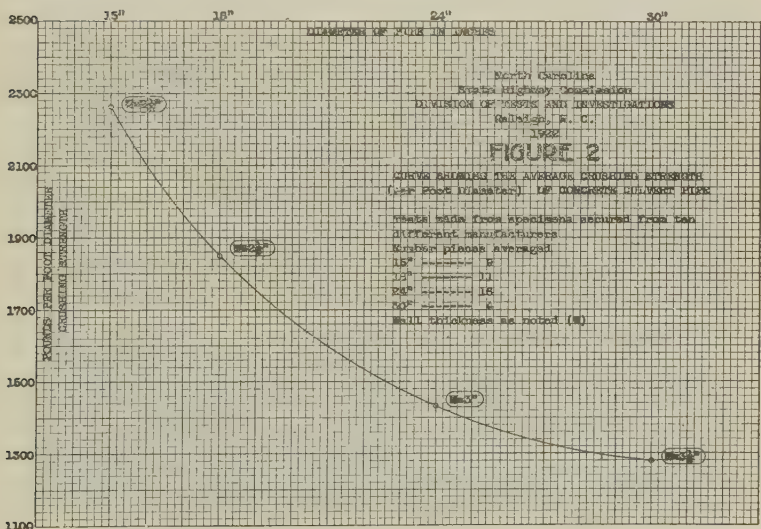


FIG. 2.—CURVE SHOWING AVERAGE CRUSHING STRENGTH OF CONCRETE CULVERT PIPE.

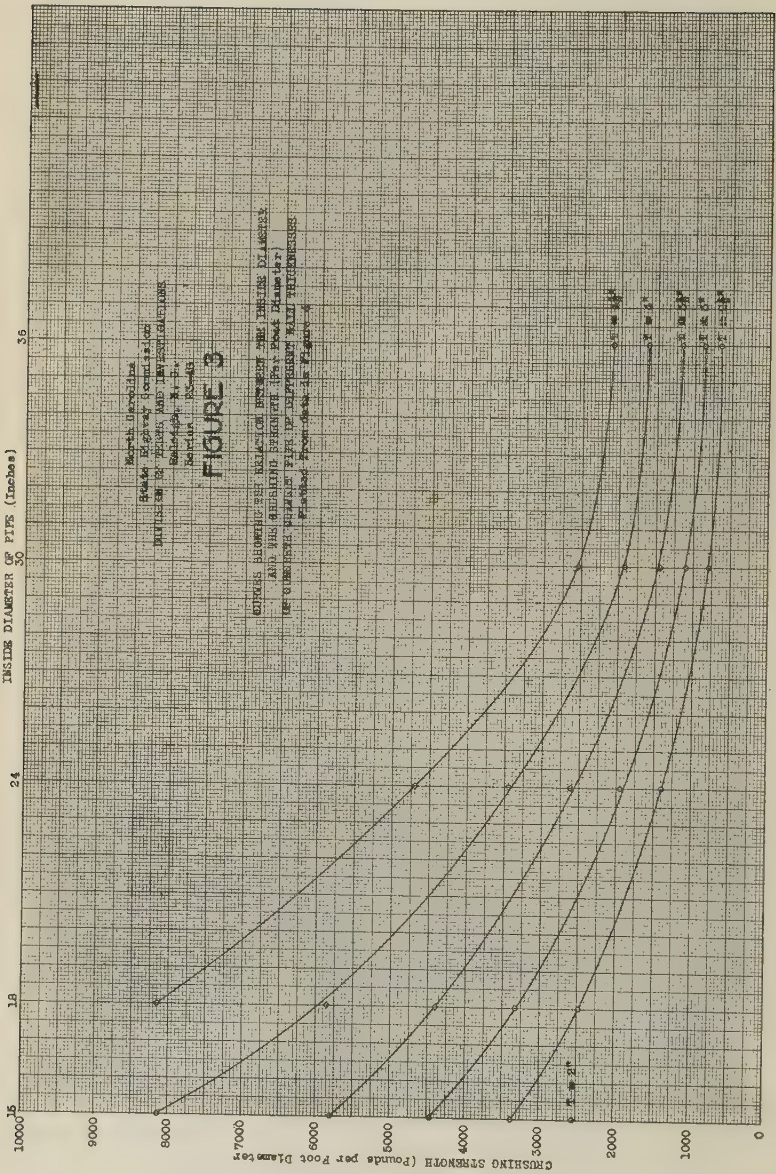


FIG. 3.—RELATION BETWEEN INSIDE DIAMETER AND CRUSHING STRENGTH.

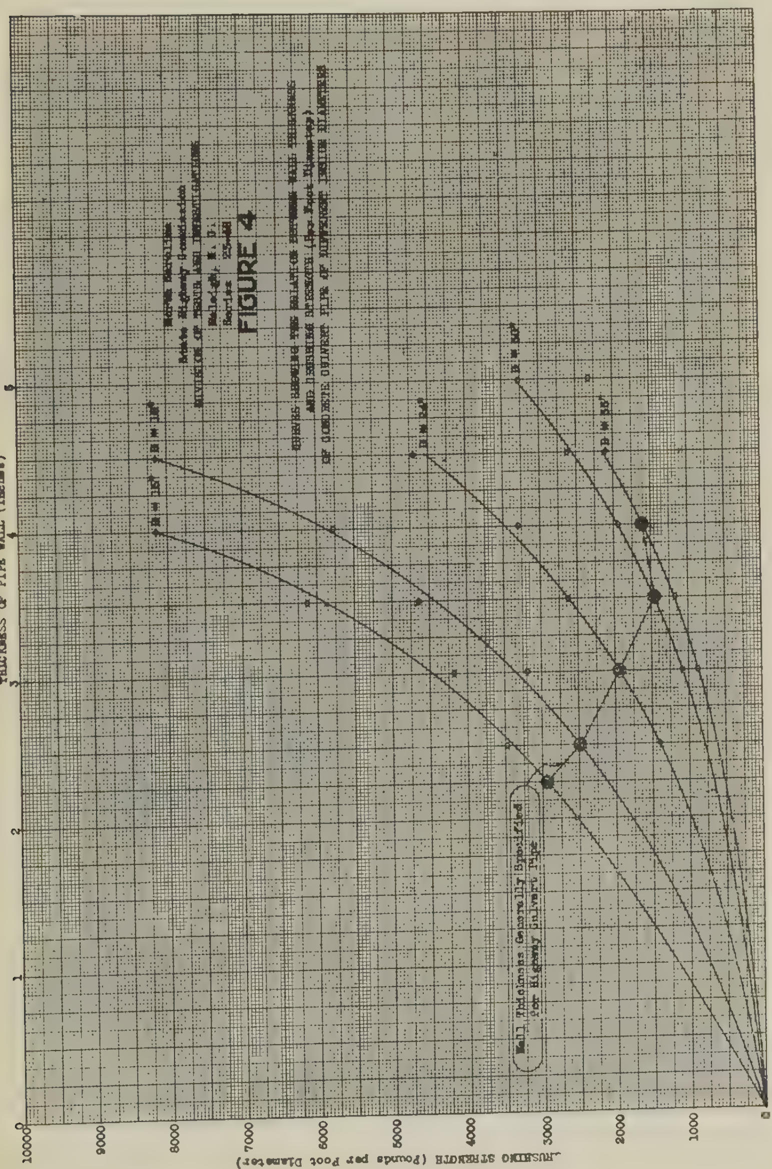


FIG. 4.—RELATION BETWEEN WALL THICKNESS AND CRUSHING STRENGTH.

After mixing, the concrete was transferred from the mixer to wheelbarrows and then placed in the forms in 4 in. layers and rodded by the standard bar used in laboratory practice. The forms were removed in 48 hours and the specimens then sprinkled daily for 25 days.

All specimens were tested at the age of 28 days with a sand bearing. With the exception of the largest size, the tests were made in a Riehlé Standard Pipe Testing Machine. Specimens too large to be tested in this machine were tested by means of a loadometer in a wooden frame testing machine. The loadometer was in turn calibrated by the Riehlé machine

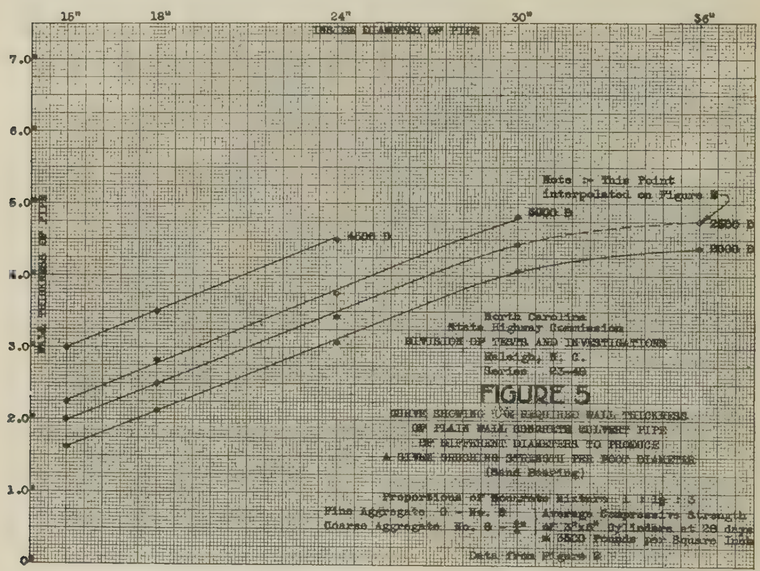


FIG. 5.—REQUIRED WALL THICKNESS OF CULVERT PIPE.

so that the results are comparable and no variation would be encountered due to a change in testing machinery. The same bearing apparatus was used in all cases.

The mix was approximately 1 : 1½ : 3 and the concrete remained for five minutes in the mixer. The fine aggregate was graded from 0 to the No. 8 sieve and the coarse aggregate from the No. 8 sieve to three-quarters of an inch in size.

The amount of each material used was as follows:

Material.	Per Cent of One cu. ft. of Material for One cu. ft. of Concrete.
Cement	27
Fine aggregate (No. 0 to No. 8)	35
Coarse aggregate (No. 8 to $\frac{3}{8}$ inch)	32
Coarse aggregate ($\frac{3}{8}$ to $\frac{3}{4}$ inch)	41

TABLE I.—RESULT OF INDIVIDUAL TESTS OF CONCRETE CULVERT PIPE OF DIFFERENT WALL THICKNESSES AND INSIDE DIAMETERS.

Diameter of Pipe, inches.		Wall Thickness of Pipe, inches.						
		2	2½	3	3½	4	4½	5
15		3,150	4,200	5,590	7,245	11,520		
		2,800	4,120	5,000	8,095	10,310		
		3,580	4,685	5,300		10,145		
				5,000		8,800		
	Aver.	3,177	4,335	5,223	7,670	10,194		
18			3,000	5,580	7,110	9,860	13,730	
			3,400	4,700	7,325	8,540	11,245	
			4,865	4,000	6,470	8,200	11,665	
				4,895		8,000	12,280	
	Aver.		3,755	4,794	6,968	8,650	12,230	
24			2,400	4,020	5,690	6,400	10,635	
			2,200	3,000	5,460	6,105	9,095	
			3,830	4,335	4,800	7,325	7,800	
				4,165	5,000		10,115	
	Aver.		2,810	3,880	5,238	6,610	9,411	
30				3,000	3,600	4,600	6,400	6,600
				2,200	3,700	5,200	5,800	11,000
				3,000	3,600	5,400	7,400	8,000
					3,600	4,200		7,000
	Ave.			2,733	3,625	4,850	6,533	8,150
36				2,800	3,600	4,800	6,800	6,000
				2,000	3,400	4,800	5,800	7,000
				3,000	3,800	5,600		7,800
					3,200	4,200		
	Aver.			2,600	3,500	4,850	6,300	6,933

Fig. 3 shows the relation between crushing strength and diameter of pipe having different wall thicknesses.

Fig. 4 shows the relation between the crushing strength and the wall thicknesses of the specimens having different diameters. The wall thickness generally specified for the different diameters is sketched in by the dotted line. It will be noted that for a given crushing strength on the

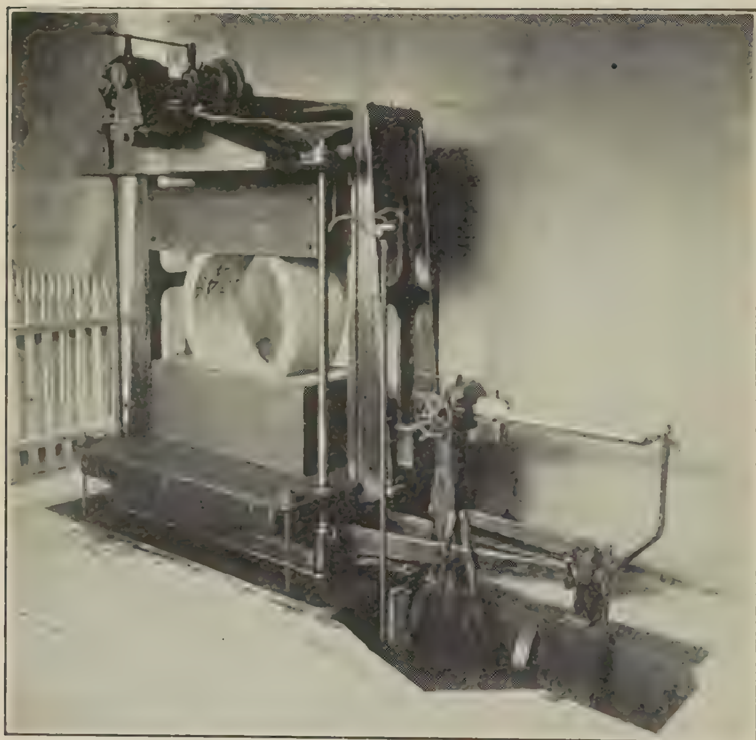


FIG. 6.—CULVERT PIPE IN TESTING MACHINE.

foot-diameter basis, changes can be made in the usual wall thicknesses and a step toward greater uniformity be taken in this manner.

Fig. 5 shows the relation between the diameter and the wall thickness of pipe for a given crushing strength. These data indicate that up to the 30 in. diameter pipe, the wall thickness is directly proportional to the diameter for a given crushing strength.

It should be borne in mind that the results of the investigation are platted with reference to the actual strengths obtained. The use of the

data given would be entirely in a relative sense unless in practice a concrete was obtained which would be identical with that used in this investigation.

The data given do not take anything into consideration but the relation of wall thickness to the diameter of the pipe on a basis of crushing strength. It would probably be necessary to depart from the relative values given for certain sizes so that allowance could be made for the necessity of having the finished product stand a reasonable amount of strain due to shipment and placing.

APPENDIX.

Considerable of the variation in the crushing strength of concrete pipe is on account of segregation of the concrete aggregate and a tendency toward porocity, due to the drier mixtures being difficult to place properly around the reinforcement, etc. The use of excess water up to a certain amount makes the concrete more workable but at the same time causes segregation and loss of strength. Any means by which the concrete could be made more workable, without decreasing the strength, would be considerably more desirable than the present necessity of using excess water to promote workability.

After the investigation to determine the relation of wall thickness to diameter of concrete pipe, another investigation was carried on to determine the effect of a diatomaceous earth, commercially known as celite, when used as an admixture to promote workability, on the crushing strength of

TABLE II.—CRUSHING STRENGTH (PER FOOT DIAMETER) OF CONCRETE PIPE WITH AND WITHOUT ADMIXTURE OF CELITE.

	Size of Pipe, inches.			
	18 x 3½		24 x 3½	
	Plain.	3 Per Cent Admixture.*	Plain.	3 Per Cent Admixture.*
	4480	4147	2730	2850
	4837	4450	2373	2395
	4350	4843	2265	2213
	3757	4125	2738	2950
	4540	4887	2780	2767
	4337	4750	2038	2320
			2175	2528
Average.	4384	4534	2443	2575
Per cent.	100	103.4	100	105.4

* 3 per cent Admixture = 3 per cent admixture of celite earth (by weight of the cement content).

the pipe. The result of this investigation indicates with an addition of 3 per cent of celite, by weight of the cement content, an average increase of $4\frac{1}{2}$ per cent in the crushing strength was obtained at the 28-day period.

The investigation consisted of tests of 26 pieces of concrete pipe; 13 pieces using plain concrete, and 13 pieces containing the celite as an admixture. Due to but one set of molds being available for each diameter and wall thickness, it was necessary to vary the wall thickness each day and correct for this change. Inasmuch as the same number of pieces of each wall thickness were made with and without the admixture, and the correction was a constant, there would be no variation due to this.

The results of this investigation are summarized in Table II.

DISCUSSION.

R. W. Crum (*by letter*).—The very excellent test results secured by the author of this paper indicate that a more general application may be made to the problem of design of plain concrete pipe than Mr. Hutchinson suggests. The following table computed from the data in Table I, showing the average moduli of rupture for the various diameters and wall thicknesses, apparently demonstrates that if the concrete is of constant quality, the moduli of rupture are reasonably independent of diameter and wall thickness, and can therefore be used as a basis for design.

The formulas for computing the moduli of rupture are discussed in Bulletin No. 36, Engineering Experiment Station, Iowa State College, and are as follows:

$$M = 0.20\ (R + \tfrac{1}{2}t)\ \frac{W}{12}$$

and

$$p = \frac{6\ M}{t^2}$$

where

M = the maximum bending movement in the tile wall in inch pounds per lineal inch.

R = the radius of the inside line of the tile shell, in inches,

W = the breaking load per linear foot, using sand bearings,

p = the modulus of rupture of the material, lb. per square inch,

t = thickness of wall.

Average Moduli of Rupture in Pounds per Square Inch, from Data in Table I.

Diameter, inches.	Wall Thickness of Pipe, inches.							Average.
	2	2½	3	3½	4	4½	5	
15	675	607	512	575	605	...	...	597
18	...	615	560	612	595	677	...	612
24	...	597	582	590	580	660	...	602
30	...	...	500	500	515	555	570	528
36	...	...	562	565	607	600	570	581
Average	675	606	543	568	580	623	570	584

Therefore, if a manufacturer knows by testing, the probable minimum modulus of rupture of his product he can design tile of different diameters to meet any specification for crushing strength. These formulae may be applied as follows:

To find wall thickness for various diameters to meet a specification requiring the strength of the pipe to be 2400 pounds times the diameter in feet, using concrete of 600 pounds per square inch modulus of rupture.

$$W = 2400 \times \frac{2R}{12} = 400R$$

$$p = 600$$

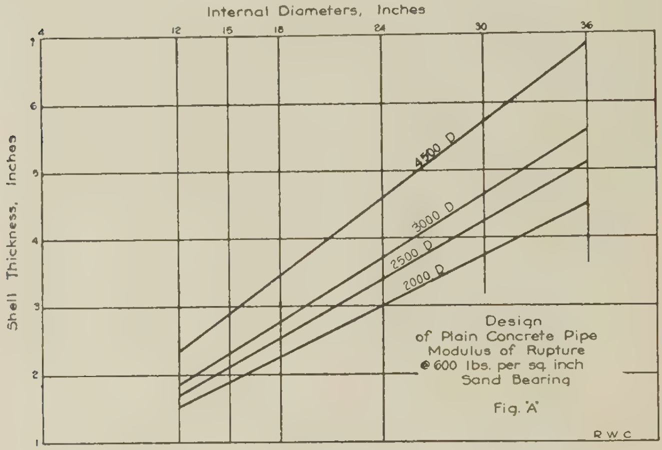


Fig. 'A'

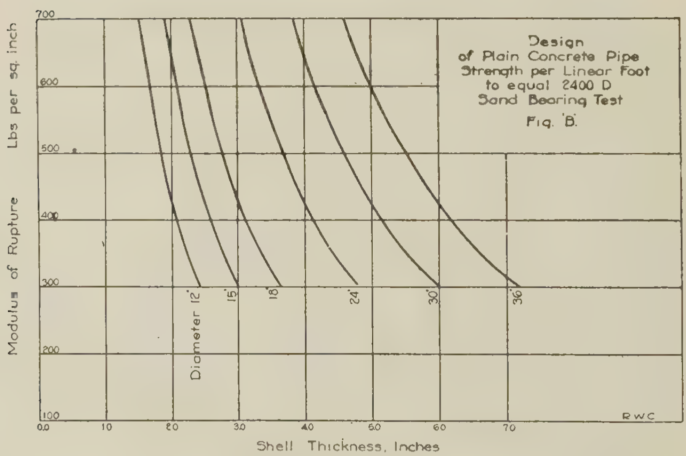


Fig. 'B'

The two formulae can be combined as follows:

$$10pt^2 = WR + \frac{tw}{2}$$

then $t^2 - \frac{tR}{30} = \frac{R^2}{15}$, $t^2 - 0.033tR = 0.067R^2$

$$(t - 0.0165R)^2 = 0.0687R^2$$

$$t - 0.0165R = .262R$$

$t = .278R$, from which the thickness of various diameters can be computed.

Solution of the formula is probably a satisfactory way of handling the problem in any given case, but for quick consultation under specific sets of conditions diagrams can be easily made. Two such are appended hereto. Figure A, gives the relation between internal diameter and shell thickness for a modulus of rupture of 600 and for various strength requirements. It will be noted that results from this diagram agree in general with Mr. Hutchinson's Figure 5. Fig. B gives the relation between modulus and rupture and shell thickness for various diameters and a specified strength under sand bearing loading of 2400 D per linear foot.

A DEMONSTRATION IN MAKING TRIM STONE AND ORNAMENTAL CONCRETE.

By R. F. Havlik

Mooseheart, Ill.

The session of the Convention was devoted to an actual demonstration of the methods used in making ornamental concrete, at the School of the Loyal Order of Moose, at Mooseheart, Ill. The following is an abstract of the discussion during the demonstration.

At Mooseheart we manufacture trimstone and building block for our own use only, and garden furniture for our own use and for a few customers such as Marshall Field & Co. and Wanamaker, and about two other concerns like that in the larger cities. The reason we manufacture garden furniture is to give the boys experience in making ornamental products. It is impossible to put enough ornaments into an ordinary building to give the boys sufficient practice in such work.

The molds for garden furniture, etc., are almost identical with those used in the manufacture of building trim. There are various kinds of processes used in the manufacture of trimstone. We use the one that seems most suitable at the particular moment for the job at hand. We use plaster, wood and glue molds. Many manufacturers have a decided preference for one type or the other. I believe the tendency among most manufacturers is towards plaster molds in preference to glue molds. On highly ornamented pieces you have to use what is known as a plaster piece mold. A glue mold will accomplish the same purpose, and if it is handled properly will do it just as well and in some respects better, as there are no seams in a glue mold except where the glue is cut into pieces to release it from the model. There may be three or four seams on a glue mold that would require 18, 20 or 30 seams in a plaster piece mold, so from the standpoint of touching up that piece of stone after it comes out of the mold, the glue mold has advantages in that there are less seams, and therefore less work involved in pointing up the stone. That applies principally to stone made by the cast method.

One other reason for using plaster piece molds as extensively as they are being used is the fact that the majority of the manufacturers tamp their trimstone instead of pouring it wet. You cannot tamp stone in a glue mold and take the mold off of the stone immediately. Stone must remain in a glue mold overnight, so if a manufacturer must use the tamp method, he is compelled to use plaster piece molds. I wish it understood that each has its particular advantages and if the manufacturer can use both the cast and the tamped process, he should use whichever of the two is the cheapest for the particular job at hand. There is some difficulty in matching tamped stone with cast stone, especially if you use white cement.

Tamped stone is likely to be darker in color than the cast stone. You cannot make it as light as the cast stone, so the only alternative is to put a little gray cement with the white to make it match the tamped stone, and take the precaution of making the tamped stone just as wet as possible; that will bleach out the resulting cast so that it can be made to match perfectly with the cast stone.

Before these boys start to demonstrate some of this work, I will show you a sample of a glue mold. That is the glue jacket for making this

DEMONSTRATION AT ANNIVERSARY MEETING DEVOTED TO TRIM STONE AND ORNAMENTAL CONCRETE.

ornament, which, you see, is highly undercut. It would be impossible to produce this in an ordinary plaster draw mold, but we could make a plaster piece mold for that particular object, but it would require a great many pieces, probably 13 or 15, whereas this glue mold is in one piece. As you see, it is flexible, and can be bent like a piece of rubber. It was made last week and has laid around the hotel here for several days. We have two unfinished concrete casts of this same ornament on the platform. You will notice these casts look exactly like this plaster cast in their physical appearance. We will dip one in acid to show you how acid affects the surface and how quickly it takes effect, while one of the boys makes a plaster cast in the glue mold. The plaster will set up in about 15 minutes, and

after it is hard enough we will release it and show you how it comes out. Another lad will spin a model of a baluster on this rigging, which is a template made of sheet metal mounted on a wood frame. A third boy will run a molding the shape of the template on this table, and if time permits we will make a pier cap of it.

Over to the left we have what is known as a plaster piece mold. The boys have just finished tamping an ornament in it of concrete. This outside ring is part of a jacket which holds the pieces of the mold in place, and within it you will see the balance of the mold made up of a number of pieces. I will let the boy take them out for you to show how they fit together. These boys are regular Mooseheart students. All Mooseheart students are required to complete a high school course and learn a trade before they are permitted to leave. When they leave we find them positions with people having use for men of their particular training. We have many calls for young men in ornamental concrete work, from our friends in the Institute, and we appreciate that very much. These pieces are made so they will draw freely from the mold. Each piece is tapered so it can be released without injuring the ornament. It takes a lot of skill—high grade skill—to manufacture a plaster piece mold properly. This same ornament could be made in a glue mold in at least half the time, and perhaps even less than that, but the ornament could not be made by the tamped method in a glue mold.

This student will now assemble the mold again and make another cast and I will have one of the other boys run a molding here and the third will start spinning that baluster, and while they are performing I will explain some of the steps of the work involved. Inasmuch as they will use the glue mold ahead of anything else, I will explain the method of producing this. A modeler must first model this ornament in clay or modeling wax. The modeler is the highest priced mechanic in an ornamental concrete plant, and is paid from $1.50 an hour up, usually up. As soon as a model is finished up, and the architect approves it, it is mounted on a table or a panel. A piece of paper is laid over the model to keep material from sticking to it; then a thin layer of clay is placed over that. That layer of clay takes the place of this glue jacket, and is later replaced with this glue.

We make it as thin as feasible, in order to use as little of the glue as possible. The next step is to make a plaster jacket to hold the glue. The plaster jacket should be so made that the glue can only fit into it in one position. This is accomplished by building up a locking ridge of clay on the layer of clay. This, of course, is later replaced with the glue. When the locking ridge has been built up on the clay, the entire surface is covered with a layer of plaster. Fiber dipped in plaster is worked into the plaster in order to make the jacket stronger. As soon as the plaster sets up, the jacket is removed from the clay. The clay is then removed from the model, the paper is removed, and the model is cleaned. The plaster jacket is then replaced over the model. A hole is cut into the top of it, as you see, and

the operator then places a funnel over the hole, sticking it in place with a little plaster. The intervening space between the plaster jacket and the model is then filled with hot glue. The glue comes in a flake form, and is practically the same as cabinet makers' glue, and can be purchased of any large packing company. The glue has to set overnight, usually, and the next morning the jacket is taken off first, then the glue jacket, and finally the model.

The glue jacket looks something like this. If it is a very complicated piece, it may be cut into three or four sections to enable one to draw it off of the model without damaging the model. The next step is to take that glue mold and dry it out with some French chalk. The model was previously greased, and this glue will be full of grease. The French chalk simply dries it up and removes that grease. The next step is to paint that entire surface with saturated alum water. That hardens the glue and also waterproofs it slightly. After the alum water dries off, the jacket is painted with litharge and linseed oil paint. The paint is allowed to dry overnight, and the entire surface of the glue is then greased with a grease made of stearine wax and kerosene. The purpose of the grease is to keep the concrete cast from sticking to the mold, and also to keep the water from soaking into the glue and rotting it. It does not accomplish this entirely, as evidenced by the fact that a glue mold is only good for seven or eight casts in cool weather, after which the glue is too soft for use, and has to be melted and the above process repeated again.

At this point I will explain how the glue is prepared. It comes in flake form. We place it in a big vessel large enough to hold whatever quantity we wish to use. We then pour water over it, drain the water off immediately, and place the glue in a double boiler and heat it until it melts. We then pour the hot glue into the mold, as described before.

The work involved in producing an object that is symmetrical consists of making a template in the first place the reverse of the profile of the object to be made; that template is then mounted on a piece of wood in the manner of this baluster template, or in this manner, for molding. We picked out the simplest kind of objects to demonstrate with because we wanted to make sure we would complete the demonstration. The next step in making one of those models is to prepare the plaster, as the boys have done now, and build up a core with anything that is available that will hold the stuff together. We use excelsior, burlap, or this material, which is known as fiber. Fiber consists of the whiskers of cocoanut shells, and is prepared by tanneries, and costs about 12¢ a pound. Fiber add strength to the molds. We could build them up of plaster only, but they would be very brittle.

A Member.—Where can these boys continue their training in mold making? You say very few become proficient in the making of molds.

Mr. Havlik.—Do not confuse what I said. Few become proficient in modeling. A boy has to have natural ability, and lots of it, and spend many, many years in the work to become a high-grade modeler. If you

question the average modeler, you will discover that he started to learn this work at about 14 years of age; he did not consider himself a journeyman until he was 20 or 21 years of age, after five or six years' experience. Nowadays they will not accept a lad in the modelers' union unless he shows some ability; he is given a short trial period and has to show considerable ability before they will admit him to apprenticeship, and then he has to spend not less than four years in the work. We just bring a boy to the point where he is a good beginner and ready for apprenticeship in modeling, but in mold making and model making our boys will fit into the average plant immediately without much difficulty.

The bigger the plant, the more mechanical methods they will use in this kind of work. They will mix all their plaster by machinery with laborers, and deliver the ready material to the casters who make the molds and models. Then again we will use clay for a parting strip. This mold is a mold for a small baluster, the same as the model the lad is spinning there; we will build up a partition of clay, at this joint, and then cast this one section of the mold. In the bigger plants they will just smear the plaster on there, run it over that joint and take that off and cut that mitre on a band saw to save time, because the time saved on the workman effects a greater saving than the loss of material in that little extra plaster that you throw over that joint, but in a school where we are limited as to expense of raw material we have to use it as sparingly as possible, so we use that clay for building those partitions, in most cases.

A Member.—Where can they get their experience in modeling? Do they have to go to plants to get that, or to schools?

Mr. Havlik.—That is the most difficult thing to answer satisfactorily. The art schools, as a rule, do not teach the commercial type of modeling; they teach a lad how to become a high-grade sculptor, modeling the human figure, and things of that sort, but building ornamentation involves very little of the human figure and the art schools touch on that only a little, and the modeler who comes out of an art school has to serve an apprenticeship in a concern that makes a business of ornamental plaster work. In Chicago the best modelers are to be found with the big plaster companies such as the Architectural Decorating Company, the Decorators' Supply Company, McNulty Brothers, and the terra cotta companies. Men who model the plaster work like that in the ceiling of this hotel are capable of doing this work.

A Member.—Is that made in molds?

Mr. Havlik.—All of that ceiling ornament is pre-cast in a mold similar to this mold. Then it is stuck up in sections and the joints are afterwards pointed up with plaster so it looks like one monolithic piece. The straight moldings are usually run in place. One reason why products manufacturers find difficulty in getting enough men who are competent to make molds of this kind, is that they do not know where to go; the best place is to stand at the timekeeper's gate of one of these big plaster concerns and grab them as they come and tell them you want men to make

good, high-grade molds for concrete work. They will know nothing about casting concrete work. You will have to supply them that knowledge, but they will be very proficient in plaster work.

If this piece just made in the plaster piece mold were made of granite instead of white cement, crushed marble and white sand, we would have to spray it with a fine mist of water through one of those atomizing sprays that fruit growers use in spraying trees. That would wash off some of the surface cement or drive it into the concrete. That method, however, makes a stone that has a milky appearance; if you wish it to look clean and glisten, you must wash it a little with acid and water after it has hardened a few days. Some will disagree with me on that, I know, but I know that when we show samples made by the two methods to visitors, almost all of them say they prefer the glistening surface to the dull surface. It costs just a little bit more to give it that final wash with acid and water. You can, if you like, let it stand just as it comes from the mold for a few days, cure it thoroughly and then dip it in an acid bath, which will not cost much more than the spraying. Just drop it in the bath and leave it there ten or fifteen minutes and take it out and wash it off with a hose and it is ready for use.

A MEMBER.—Do you keep time on it?

MR. HAVLIK.—Some time, but usually the time is determined by the appearance of the stone.

A MEMBER.—How proficient are your graduates, and how much are they paid?

MR. HAVLIK.—That is a good point to bring up. If you are fortunate enough to have me send you one of our boys, be patient with him for a few days, as he may lack commercial speed which he can only get by working 8 hours a day every minute of the day. No student was ever born who exceeded the speed limit working in a school, and these boys are just like you were when you were 15 or 16. They have to get that speed in a plant, so if you happen to get one of them sometime, bear that in mind and give him a chance to become accustomed to your methods and the speed of your shop; as soon as he gets acclimated, he should make good; if he does not, let him out and let him look for a job somewhere else; that will make a man of him quicker than any other treatment. A fair chance is all we ask for any of these boys. One of our first graduates who graduated in 1920, is completing his senior year at Wisconsin University; last summer I secured a position for him at Nashville, Tenn., to turn out a sunken garden for the Peabody College. The President of the college wanted me to come down and tell them how, but I advised him I could send someone to do the complete job, and they accepted my suggestion and turned the whole job over to this young man. He finished it up in about nine weeks' time and they paid him $55.00 a week for that work. We have sent boys out that we started at 75¢ an hour, but they paid this young man at the rate of $1.25 an hour; a dollar and twenty-five cents is probably the highest rate of pay, and seventy-five cents is the lowest. We

hope they will all be worth more than 75¢ after they are in a commercial shop. If they are not, we are not making good at our job.

Here is the plaster cast that the young man just released out of that glue mold. If this were concrete he would have to wait 24 hours before he could take it out.

While they are finishing up those other jobs, we will have one of the young men dip this ornament, which is made of cement, in this acid bath, and we will time it and see how long it takes to make a job.

A Member.—How strong is that acid?

Mr. Havlik.—About 3 parts of water to 1 part of commercial muriatic acid. I think we usually use about 4 parts of water to 1 part of acid. There is such a thing as getting it too strong for practical purposes—it may etch the stone out more rapidly than the average workman will handle the cast. We keep adding a little acid as it weakens. We dip almost everything at Mooseheart, and I think we will renew the acid entirely perhaps once every two years. It lasts a long time when fresh acid is added to it at intervals, but ultimately you fill up the lower part of your tank with a lot of dead cement, and are wasting space, so you have to get rid of that sooner or later. This cast has been in there just two minutes, and you can see some of the granite exposed already. The lad has rubber gloves on. In our plant we do not use rubber gloves at all because they do not get their hands in often and it does not hurt unless you have a cut on your hand.

A Member.—How do you prevent air bubbles?

Mr. Havlik.—We do not prevent them 100 per cent, but we eliminate them perhaps to 5 or 10 per cent by jarring the molds and taking special precautions to press the facing mixture into every part of the mold as thoroughly as possible before we finish filling it up. That comes largely from experience. It is pretty hard to describe in words just how we do that, but if you will visit our plant as you are at liberty to do at any time, we can show you better than we can describe it in words.

A Member.—Do you make balusters the way that young man is making this one out of plaster by using a concrete mix?

Mr. Havlik.—No, we do not, but there are some people in the business who not only spin balusters but make columns by a method somewhat similar. It is a centrifugal method. I think they revolve the mold at a high rate of speed and then the concrete is thrown against the mold.

A Member.—Do you steam cure this stone?

Mr. Havlik.—We steam cure all stone, whether it is block, trim stone or garden furniture; we steam cure it for two to three days, and frequently ship in carloads when the stone is two days old, if it has been steam cured that length of time.

A Member.—How do you get the moisture in your steam kilns?

Mr. Havlik.—We run live steam into the kilns at an initial pressure of about 10 lb. to the square inch and give the kilns all the steam they will absorb. We make no attempt to run the steam through water as is

done in some plants. We get the temperature up as high as possible. We have made tests with thermometers at the ends of the kilns and found that our temperatures ran as high as 185 deg. F.

A MEMBER.—Do you have much trouble with the dropping of the condensed water?

MR. HAVLIK.—We do to a slight extent. We are very fortunate at Mooseheart in having about the right pitch to our roofs, and in the kilns that have a smooth ceiling we have practically no trouble from the dripping, but I have noticed in another plant where they have a wood roof that the condensation collects in the joints between the lumber, it frequently drips where you don't want it to drip and causes a hole in the stone below it. We plaster our ceilings over metal lath and trowel them smooth, and that gives very satisfactory results. We pitch them about a foot and a half in 8 ft.

A MEMBER.—How long do molds last?

MR. HAVLIK.—A plaster mold is good for at least 40 or 50 castings with ordinary care, and if you take very good care of the mold, it can be made to last several years, but you have to repair any corners that happen to be knocked off. A glue mold is good for 7 to 8 castings in cold weather, and 3 or 4 in hot weather. The glue is not destroyed, but is softened so it has to be remelted and you usually have to add a little fresh glue to it. The loss in glue is very small.

The question has been asked if we paint our glue molds; I forgot to mention that; we do. Manufacturers used to be very secretive about what they used to paint a glue mold to waterproof it. Some prefer white lead and oil. We think that we get better results by using litharge and linseed oil, but either will give you a good surface. The mold has to set for about 24 hours after it is painted with this litharge paint or white lead paint. The question has been asked as to how you handle big stones and dip them in acid baths. We use a bath 4 x 4 x 13 ft. and handle our big stone with hoists, and either handle the stone directly with that hoist into the tank or use a rack, depending on the kind and size of the stone to be dipped. It is not very expensive. I'd rather make a great big stone and dip it in acid than make stone by any other process; it is more profitable, as the cubage runs up very rapidly.

A MEMBER.—Have you ever tried to sand blast anything?

MR. HAVLIK.—No, we have never done that. It ought to work satisfactorily.

A MEMBER.—Do you scrub any at all?

MR. HAVLIK.—Yes. In dipping in an acid tank, we usually have to scrub part of the stone, as one part may have a little more cement on the surface than another part and the acid will not take effect as rapidly on that as the other. If you leave it in the tank until that portion that has more cement has etched out satisfactorily, the rest may be etched a little too much, so you must watch that and scrub such parts with a scrubbing brush. We spray tamp stone with a fine water spray, but after such stone

has cured we wash it off with a weak solution of acid and water to brighten it up. You have noticed in the exhibit in the other room we have some panels that have two textures, such as the heads of Martha and George Washington; the heads themselves are made of white cement and marble, and the background is made of gray cement and Crown Point Spar. Two different textures were used, but when the casts were first made they looked like these casts before they were dipped. After they were dipped, one showed up with a dark background and the other with a white foreground, but that is not made by dipping one part and not the other—it is made by using two different aggregates.

A MEMBER.—What proportion of a mix is used in this casting?

MR. HAVLIK.—I am not sure as to the exact proportions of the cement and granite. This boy says it is 2½ parts of granite to one part of cement. This is experimental work. On our commercial work we use 1 part of cement to 3 of granite or other aggregate, the reason being that we keep the proportion of the cement down as much as possible to prevent checking; we also refrain from using a very fine aggregate containing dust, for the same reason. This boy has shown you the one that was dipped for a few minutes. We have another piece just like it, and will dip that too, and also a little ornament called the Dying Lion of Lucerne, a copy of a famous piece of sculpture in Switzerland.

Here is the model of a baluster ready for the top and bottom. Those parts have to be made by hand and stuck on there with plaster. This mold is the plaster mold for this same baluster.

A MEMBER.—What sized aggregate are you using there?

MR. HAVLIK.—We use the two small sizes, 3½ and 4, equal parts. We saw up that molding with a miter, just as a carpenter would saw up lumber, only not as accurately, because if we saw off an inch too much, we can put it back on there with fresh plaster, while the carpenter cannot do that. The plaster we use is No. 1 molding plaster, manufactured by the U. S. Gypsum Company. If you ever need any for this work, do not accept a substitute. It is positively the best, works the best, and in our opinion is the best for molding. There are other plasters on the market that are as good or slightly better for making models and moldings, but the U. S. No. 1 Gypsum plaster stands up best for mold work. A mold before it is used for concrete work is soaked in water long enough to kill the suction, maybe half an hour or an hour, and this plaster has to stand water.

A MEMBER.—Do you ever use Keene Cement for making molds?

MR. HAVLIK.—We have done so, but I am not in a position personally to tell you whether it is better or worse.

OLD AND NEW METHODS OF CONSTRUCTING CONCRETE BRIDGES.

By J. B. Hunley.*

The C. C. C. & St. L. Ry. abandoned the use of stone masonry for bridge construction about 25 years ago, and although some concrete was placed in the early nineties, it was not used to any great extent until 1902, at which time there was inaugurated a comprehensive program of reconstruction and extension of lines, which has since been carried on, from year to year, almost uninterruptedly.

During this period there has been placed in bridge structures about 1,250,000 cu. yd. of concrete, with all of which the writer has had something to do, in the way of design, construction, or maintenance, or perhaps all three. Some of this concrete is very good, a little is bad, a good deal of it is just fair, and it is of some interest to see under just what conditions it was placed, at different periods.

The first thing in the way of a specification extant, is a typewritten sheet covering the construction of a 14-ft. arch in 1898. The mixes only are specified, and are as follows:

Footings.—One part Louisville cement, 2 parts sand, 5 parts gravel.

Abutments and Wings.—One part portland cement (Buckeye or Alpha), 2½ parts sand, 6 parts screened gravel or broken stone.

Arch Ring.—One part Alsen portland cement, 2 parts sand, 5 parts clean, screened limestone chips.

Parapet Walls.—One part Alsen portland cement, 2 parts sand, 6 parts stone.

All exposed surfaces were to be faced with mortar—1 part cement, 2½ parts sand, 2 to 3 in. thick, using the same brand of cement as used in the abutting concrete.

It is not known why a different brand of portland cement was specified for the various parts of the structure, unless each brand was supposed to have some especial quality. This structure is in good condition, and it was cheap, costing only $1468 in labor and materials to build 366 cu. yd. of concrete; that is, $4 a yard.

---

* Engineer Bridges and Structures, C. C. C. & St. L. Ry., Cincinnati, Ohio.

The first General Specification was issued in 1902, entitled: "Specifications for Concrete Monolithic Construction." It established a proportion of 1: 2: 4 where natural cement was used in foundations, and for portland cement concrete the proportions were 1: 2½: 5 for piers and abutments, 1: 2½: 4 for arch rings and girders, and in especial cases where maximum strength was not needed, 1: 3: 6, and contained this clause: "It is understood that the proportions of stone and gravel are approximate and may vary with the character of the materials, and must always be such that the mortar will fill all the interstices in the stone when rammed," and as to placing:

"The concrete shall be put in in layers at right angles with direction of pressure in the structure, not exceeding 6 in. in thickness, and thoroughly rammed with iron rammers, and shall be of such consistency that the water shall just flush to the surface when rammed."

Such a specification, literally enforced, would today produce a very fair concrete.

The use of natural cement was discontinued in 1904, and a new specification was issued in 1905. Quite a few structures of re-inforced concrete were built in 1903 and 1904; and wetter mixtures were advocated, especially for re-inforced work and this issue required as to consistency that: "The concrete shall be of such consistency that when dumped in place it will not require much tamping. It shall be spaded down and tamped sufficiently to level off, causing the water to rise freely to the surface, but not materially above it." The proportions were arbitrarily fixed as 1: 2: 4; 1: 3: 6, etc., and the use of a premixed aggregate was permitted.

They were again reissued in 1910. The specification for cement was revised, and that for the aggregates was slightly modified, but that portion referring to consistency was unchanged.

The use of the tower and chute came into quite general use, with which the mixes became still wetter, if possible, and the specifications of 1915 were quite fashionable in requiring that: "The concrete shall be a wet mixture of such consistency that it will flow freely on a slope of 4 horizontal to 1 vertical. It shall be spaded down and leveled, causing the water to rise freely to the surface, but not materially above it."

All of these specifications were in line with general practice, but the results were not entirely satisfactory. There was no question as to the quality of the raw materials, as they were usually furnished by the railway company, the cement being inspected and the aggregates, washed and screened, from our own pits, but with the same materials and the same labor, and with conscientious inspection, it seemed impossible, to determine in advance, as to just what sort of concrete would be found when the forms were removed. Concrete in one structure would cost so much per yard and be very good, while concrete in another structure, costing identically the same per yard, would be inferior; that is, we were never certain of getting our money's worth. It was quite a gamble, and sometimes their stakes were very high.

When any of the concrete began, after a few years, to show signs of deterioration, the trouble was usually ascribed to frost action, but in many instances it was found the structure had been built in the most favorable weather. Of course, we now know that most of it was poured too wet, as was customary, but at that time this was ignored, and suspicion was diverted to the ingredients, and their proportions.

The question of proportioning caused us no little concern. Sand and stone were ordered of certain maximum and minimum sizes, but when they were combined in the fixed proportions of 1 to 2, the mixture worked well, only when we happened to have a sand from one particular pit, and stone from another. Usually it was too coarse, lacking in mortar, and it was impossible to get a good surface finish, in which case the inspector, if he were wise and experienced, would use more sand and say nothing about it, or in the leaner mixtures, he would frequently use more cement, which rarely resulted in criticism, as it was furnished free to the contractor; and his boss seemed to feel easier if a little excess cement had been used.

It was known that the mixed aggregate was usually improperly graded, and to remedy this feature, to some extent, serious consideration was given, regardless of the inconvenience, to the question of using three aggregates of different sizes, instead of two, and then combining them in such proportions as would produce an easily worked concrete, and this would have been a step in the right direction.

In 1919 the Structural Materials Research Laboratory of Lewis Institute issued a Bulletin on "Design of Concrete Mixtures" by Professor D. A. Abrams. This information was particularly interesting as it contained a sort of a promise, that, if we would break every rule of our specifications which referred to the mixing, we might expect, with some certainty, a concrete of a predetermined strength, and of uniform quality. It, of course, advocated the use of a minimum amount of mixing water, and offered a means of grading and proportioning the aggregates, which, if practical, would overcome one of our greatest difficulties.

It was not clear as how this method of designing mixtures could be adapted to construction work, but in an endeavor to accomplish something, the specifications were revised in 1921 with this clause as to consistency:

"The concrete shall be of such consistency as, and shall contain no more water than, is necessary to permit of proper working. It shall be spaded down, leveled, and tamped, causing the water to rise to the surface. If water accumulates on the surface of the concrete, the amount of water used in the mix shall be reduced."

The minimum time of mixing was specified as 45 seconds, and when towers were used it required that, "the chutes be steep enough to make the concrete run freely without using an excess amount of mixing water, but not steep enough to cause segregation of the stone and mortar."

It seemed unwise and impracticable at that time, to make any change as to the aggregates. They were, in nearly every case, furnished by the railway company, and it was thought that a better grading might be

arranged for at the pits, and the proportions of sand and stone varied on each job, as might be found necessary.

In 1921 we were back just about where we started in 1902. Making concrete is probably one of the few arts in which, during this same period, there has been any retrogression.

In the fall of 1922 work was begun on a reclocation of line and grade revision, near Sidney, Ohio, which involved the construction of about 55,000 cu. yd. of masonry, half of which was in one structure over the Great Miami River. This bridge consisted of three 140-ft. and two 100-ft. arch spans about 95-ft. high, and considering its importance and the expense involved, it was felt that it was necessary to use every means possible to get a good quality of concrete. A considerable volume was concentrated in one structure, and although but little was known as to the application of laboratory theories to field practice, it was felt that it was a good time and place to experiment with the methods of proportioning concrete, as outlined by Professor Abrams, and it was so arranged.

This work has been described in some detail (See *Engineering News-Record*, Oct. 11, 1923, and *Railway Review* of Nov. 17, 1923), but some of the more important features may be briefly reviewed.

Desired strengths, as shown below, were selected for various portions of the structure. It was realized that they were somewhat higher than would customarily have been obtained, but as the work was largely experimental, it was well to be on the safe side.

	Lb. per sq. in.
Footings	2000
Piers and abutments	2000
Arch rings	3000
Spandrel arches and superstructure	2500

In the footings, piers, and abutments, below the haunches, aggregating about 10,000 cu. yd., a premixed aggregate was used of size 0 to 1½ in. This material was run of pit gravel, washed and graded to some degree by the addition of stone. The fineness modulus varied from 5.3 to 6.7, and this wide range was due in part to lack of refined methods of grading at the pit, and somewhat to the method of handling at the stock pile. The gravel was unloaded from cars by a clamshell, resulting in a certain amount of segregation as the larger pebbles rolled down to the bottom. The hoppers from which the tram cars were loaded, were in a tunnel under the center of the pile, and often they would receive largely the finer material. As a crater was formed over the hoppers, it was filled from other cars shipped in from the pit, in which case the coarse particles did not escape, or by the coarse material rehandled from the circumference of the pile.

It was found with this material that, to avoid clogging the chutes, an average consistency, as represented by a slump of 6 to 7 in., was required. At times it could be worked somewhat drier, but due to the wide and frequent variation in the fineness modulus, it was difficult to control either

the true mix or slump. Of some 80 samples, for test specimens, taken from the mixer, the average slump was 6¾ in.

For the arch rings and superstructure, separate aggregates were used; washed sand, 0 to ¼ in., and crushed boulders ⅜ in. to 1½ in. The fineness modulus of the fine and coarse aggregates remained quite uniform, averaging 3.2 and 7.8 respectively. The batches were 28 cu. ft., and the sand and stone were mixed in such proportions as to give a fineness modulus of the mixture, varying from 5.7 to 6.0, depending upon the desired strength.

It was found that the concrete made of the separate aggregates could easily be handled in the chutes and forms with a slump of 3 to 4 in. The average slump of some 300 samples, representing 18,000 cu. yd. of concrete, was 3½ in. This produced a concrete much drier than it was thought could be handled through chutes, and much drier than the gravel concrete previously placed with the same equipment. This seemed difficult to explain, as all materials were from the same pit, but it was undoubtedly due to the fact that the size and grading of the sand and stone remained nearly constant, and that the proportions of each were such as to produce a very workable mixture. All concrete was mixed for a minimum time of 1 minute.

Unless it is certain that the fineness modulus of a premixed aggregate has a proper value and will remain fixed, there seems but little justification for its use.

The concrete was conveyed to the forms by means of a tower 140-ft. high, and chutes having a pitch of 3 and 2½ horizontal to 1 vertical, and with these responsible slopes the dry mixtures, and many batches had slumps of only 2 to 2½ in., were successfully handled without clogging or segregation. Considering that there were some 20,000 yd. of comparatively dry concrete so placed, it is quite evident that concrete, properly proportioned and mixed, can be handled by such means, without using an excess amount of water, as has been so customary and which has brought the tower and chute into general disrepute. In fact it was found that, with thorough mixing, less difficulty with clogging was experienced with a reasonably dry concrete than with the wet mixtures.

The surface of the finished concrete is of excellent texture and quality. Of the many thousand square yards of surface area, there probably was not more than a total of 1 sq. yd. of honeycomb. No rubbing or washing of any kind has been permitted, except on the ends of the arch rings, where the board marks were partially rubbed out.

Experiments had clearly indicated that by regulating the mix, the consistency, and the grading of the aggregate, a concrete of predetermined strength could be produced in the laboratory, but there was some question as to whether, by means of the crude facilities for control at hand, approximately similar results could be obtained under field conditions. As a check, 6 x 12 test specimens were made from time to time, in accordance with the A. S. T. M. Standards, except that the samples were drawn

directly from the discharge of the mixer instead of from the forms, so that the actual slump of the particular batch could be determined. These specimens were buried in damp sand and tested at the end of 28 days. The early tests gave erratic and unsatisfactory results, but it was found that the cylinders had been improperly capped and cured, and when this was corrected the strengths obtained closely approximated the designed strength.

A summary by groups with various fineness moduli and mixes, representing 280 tests taken from the Miami River bridge, is shown in Table I. As stated, the samples of concrete were taken from the mixer and the slump measured. The batches were of the regular run with no refined proportioning or measurement as would occur in the laboratory. In many cases it is known that the amount of cement varied between batches as much as 10 per cent; as in some of the mixes where for instance 5½ sacks were required, 5 sacks would be used in one batch and 6 in the next. This, of course, gave the average mixture in the forms, but might materially affect the strength of the specimens. The test cylinders were stored out of doors and were subject to all of the variations in temperature, and considering the comparatively crude methods employed, it is not surprising that there was considerable variation in the strength of the individual specimens from

TABLE I.—BRIDGE NO. 199, SIDNEY, 1923.

Average results of test groups: 28-day test, 6 x 12 cylinders, 280 tests.
Aggregate: Sand, 0 to 4; fineness modulus, about 3.2. Pebbles and crushed boulders, ⅜ to 1½ in.; fineness modulus, about 7.8. Time of mixing, 1 to 2 min.
Concrete for specimens taken from mixer.

Fineness Modulus.	True Mix.	Slumps. 2 to 2¾ in. Number of Tests.	Expected Strength.	Actual Strength.	3 to 3¾ in. Number of Tests.	Expected Strength.	Actual Strength.	4 to 4¾ in. Number of Tests.	Expected Strength.	Actual Strength.	5 to 5¾ in. Number of Tests.	Expected Strength.	Actual Strength.	6 to 8 in. Number of Tests.	Expected Strength.	Actual Strength.
5.60	1–4.45	11	2610	2450	..	....	....	..	....	....	..	....	....	..	....	....
5.70	1–3.66	..	....	....	4	3050	2978	10	2822	3577	6	2710	3398	..	...	....
5.80	1–3.12	..	....	....	..	....	....	6	3320	3594	..	...	...	..	...	....
	1–3.38	..	....	....	..	....	....	..	....	...	6	2920	3210	..	...	....
	1–3.68	12	3235	3221	..	....	....	10	2920	3364	6	2740	3628	..	....	....
	1–4.05	22	3000	2810	..	....	..	..	...	...	..	....	...	..	....	....
	1–4.50	..	....	....	6	2570	2490	..	....	....	..	....	....	..	...	....
5.85	1–3.12	6	3550	3494	18	3390	3737	5	3320	3643	6	3110	3873	..	....	....
	1–3.38	12	3405	3527	36	3240	3090	..	....	....	..	....	....	..	...	....
	1–3.68	..	....	....	6	2990	2205	..	....	....	..	....	...	..	....	....
5.90	1–4.09	3	2970	2414	9	2900	2913	2	2750	3472	..	....	...	..	...	....
	1–4.55	7	2730	2500	31	2615	2395	14	2485	2506	..	....	....	..	...	....
6.00	1–2.36	..	....	....	2	4110	4186	..	....	....	..	....	....	4	2840	2203
	1–3.43	..	....	....	12	3310	3400	..	....	....	..	...	....	..	....	....
	1–3.62	..	....	....	8	3225	3066	..	....	....	..	....	....	..	....	....
Weighted Aver...		73	3080	29.2	132	3062	2980	47	2840	3215	24	2870	3520	4	2840	2203

the expected strength. However, when a number of results are grouped and the average taken, the strengths closely approximate that expected and are probably quite representative of the concrete actually in the structure.

There is also included, in Table II, the results of 132 tests made at Beech Grove in connection with the manufacture of concrete crossing plank. They may be of interest, as we were working there with dry and very rich mixtures. The high slumps shown were used only for a special series of tests, as the concrete actually used in the slabs had a slump of about 2½ in.

This method of proportioning concrete was used not only at the Miami River bridge, but at all the structures of importance involved in the work, aggregating about 40,000 cu. yd. Some 800 tests have been made to date and the results are similar to those tabulated.

All of the concrete is of excellent quality and the use of these refined methods and drier mixes seems to have inconvenienced the Contractor but

TABLE II.—CONCRETE CROSSING PLANKS MADE AT BEECH GROVE, JUNE–NOVEMBER, 1923.

Average results of group tests: 28-day test, 6 x 12 cylinders, 132 tests representing 800 slabs.
Aggregate: Washed Sand, 0 to 4; fineness modulus, about 3.25. Crushed limestone, ⅜ to 1 in.; fineness modulus about 7.25. Time of mixing, 1½ to 3 min.

Fineness Modulus.	True Mix.	Slumps.														
		0 to 1¾ in.			2 to 2¾ in.			3 to 3¾ in.			4 to 4¾ in.			5 to 5¾ in.		
		Number of Tests.	Expected Strength.	Actual Strength.	Number of Tests.	Expected Strength.	Actual Strength.	Number of Tests.	Expected Strength.	Actual Strength.	Number of Tests.	Expected Strength.	Actual Strength.	Number of Tests.	Expected Strength.	Actual Strength.
5.60	1–2.59	..	....	....	16	3845	4090	..	....	....	..	....	....	..	...	....
5.70	1–2.63	..	....	....	6	3830	4398	..	....	...	..	....	....	..	....	....
	1–2.85	2	3800	2630	4	3680	3060	2	3520	3375	..	....	....	..	....	....
5.85	1–2.16	..	....	....	..	....	....	..	...	....	8	3780	3978	..	....	....
	1–2.52	..	....	....	..	....	....	8	3810	3550	..	....	....	..	....	....
	1–2.72	..	....	....	8	3840	3970	..	....	....	..	....	....	..	....	....
5.90	1–1.91	..	....	....	..	....	....	..	....	....	4	3960	4317	..	....	....
	1–2.18	..	....	....	..	....	...	4	4100	3382	..	...	....	..	....	....
	1–2.37	..	....	....	4	4140	4228	..	....	....	..	....	....	..	....	....
5.95	1–1.64	..	....	....	..	....	....	..	....	....	..	....	....	4	4160	3440
	1–1.82	..	....	...	..	...	...	4	4340	4245	..	....	....	..	....	....
	1–2.37	..	....	....	4	4200	3666	..	...	....	..	....	....	..	....	....
6.00	1–1 65	..	....	....	..	....	....	8	4510	4545	..	....	....	..	....	....
	1–1.92	..	....	....	8	4510	4500	..	...	....	..	....	....	..	....	....
	1–2.89	..	....	....	16	3854	4080	12	3706	3632	..	....	....	..	....	....
6.10	1–2.35	..	....	....	2	4210	4308	4	4150	4520	..	....	....	..	....	....
6.15	1–1.87	..	....	....	..	....	....	4	4500	3488	..	....	....	..	....	....
Weighted Aver...		2	3800	2630	68	3960	4070	46	4050	3840	12	3820	4080	4	4160	3440

little, if at all, and they have been applied to practically all of our other structural concrete work, with similar results, which has since been undertaken.

We have frequently been asked, if the application of these methods does not result in the use of more cement. It is true that we used on this work a little more cement than our old specifications would have required, but we are getting, we believe, not only a stronger concrete, but greater strength per barrel of cement.

Under the usual specification there would have been placed about 10,400 cu. yd., mixed in proportion of 1: 3: 6, and 17,600 cu. yd. of 1: 2: 4 concrete. The F. M. of the mixed aggregate, for both classes, would have been about 6.25, and as usually mixed the concrete would have had a slump of at least 7 in. The True Mix for the 1: 3: 6 concrete would be 1 to 6.3, with an expected strength of 1400 lb. per sq. in., and there would have been used 1.08 bbl. of cement per cu. yd. of this concrete. For the 1: 2: 4 mixture, the equivalent True Mix would be 1: 4: 2, with an expected strength of 2200 lb., and there would have been used, 1.52 bbl. per yd. The total net amount of cement used then, would have been 37,990 bbl. With the fixed proportions of sand and stone, the mixed aggregate would have been too coarse to get a good finish, especially on the 1: 3: 6 concrete, and the inspector would probably have used some excess cement, but this will be neglected.

For the purpose of comparison, let us multiply the strength in tons per sq. in. by the cu. yd. of this strength, and call the product "yd-tons." Then we would have had in this structure 26,640 yd. tons, or about 0.7 yd.-ton per bbl. of cement.

As actually constructed we have, using 41,670 bbl. of cement:

7,700	cu. yd. of 3000 lb. concrete or	11,550 yd. tons
9,900	" " " 2500 " " "	12,375 " "
10,400	" " " 2000 " " "	10,400 " "
28,000	" "	34,325 " "

or 0.82 yd. tons per bbl. of cement. There has been no deduction made in quantity of cement, for concrete spilled and wasted, and the strengths shown above are the designed strengths, but, as a matter of fact, the 2500 and 3000 lb. mixtures were overdesigned about 5 per cent. This correction gives us about 0.85 yd. tons per bbl.

Had we been satisfied with 1400 lb. and 2200 lb. concrete which would have presumably been obtained under the arbitrary specification, and mixed the concrete by the same methods and slumps actually used, the True Mix for the 1400 lb. concrete would have been about 1: 7.2, requiring 0.94 bbl. of cement to the yd. of concrete, and for the 2200 lb. concrete the True Mix would have been 1: 5.3, requiring 1.25 bbl. of cement per yd., or a total of 31,800 bbl. The yd. tons would be as before, 26,640, or 0.84 yd. tons per bbl. as was actually obtained in the structure.

Then it is apparent that about 6200 bbl. or 16 per cent of the cement would have been saved, at the same time obtaining concrete of equal strength, by simply controlling the water used in the mix, and varying the proportions of sand and stone, instead of making the concrete by the old "hit or miss" method. And also, that by using 2700 additional bbl., or about 7 per cent more cement, we actually obtained, with proper mixing, concrete about 23 per cent greater in strength.

With the necessary information and an inspector or engineer who takes enough interest in his work, there is nothing especially difficult about designing a concrete mixture, and after it is designed and made, nothing unusual is involved in placing the material in the forms. Efficiency in making the sieve analysis and slump tests is easily acquired, and if the inspector possesses the "Concrete Sense," he soon begins to understand what it is all about and why, and is then able to intelligently make the necessary adjustments in proportions, as occasion may demand. It is not necessary, if one have confidence in these methods, to make the test specimens, but after all, there probably is as much justification for making them, if the principle of concrete mixture design is accepted, as there is for analyzing and testing each heat of steel at the mills. It is quite a satisfaction to know the actual strengths obtained, and the expense is not great where a laboratory is available.

However, when this work was first undertaken we encountered some difficulties. It was quite a new thing and there seemed to be but little information available co-relating laboratory with field methods. We could easily combine the sand and stone in certain proportions and control the water used by the slump test, but we did not know just how much cement to use with a certain volume of sand and stone, measured as they came to us, to produce the desired True Mix as referred to in the charts and tables of strength.

In the laboratories a certain volume of cement had been used with a certain volume of dry, rodded aggregate, producing a concrete of a certain strength. This ratio of volumes was called the True Mix, a purely fictitious ratio, as far as we were concerned, as we had to deal with sand containing various amounts of moisture, and the sand and stone were measured while in a loose condition, neither one nor the mixture of the two, being dry or rodded. The volume of cement used with a certain volume of sand plus a certain volume of stone, was called the Nominal Mix, but the ratio between the True Mix and Nominal Mix was unknown to us.

We assumed, from time to time, various values for this ratio, obtained by rather crude methods, and went on with fair success, as indicated by our test specimens, but inasmuch as the determination of the factor, relating the True and Nominal Mixes, was really the answer to the entire problem, tests, by which the value of this factor might be established, were made with the various aggregates used. The results were difficult to explain until a theoretical solution was found. They may be of interest to those who contemplate the use of these methods, but perhaps are well known to those who have experimented in the laboratories.

Our problem then is to find the relation of the True Mix to the Nominal Mix, or of the True Volume of Aggregates to the Nominal Volume of Aggregates. For example, if we were to take 4 cu. ft. of sand, measured loose, and containing moisture, and 6 cu. ft. of stone, measured loose, just as we would find them on the work, the Nominal Volume would be 10 cu. ft. If the sand were dried, mixed with stone, and the mixture rodded, the resulting volume (True Volume) would be less than 10, say 7.0 cu. ft. Then for this particular mixture the ratio of True Volume to Nominal Volume would be 7.0 to 10, or 70 per cent. If 2 sacks of cement were used in such a batch, the Nominal Mix would be 1 : 5.0 and the corresponding True Mix 1 : 3.5. This ratio of True Volume to Nominal Volume, or True Mix to Nominal Mix, which we will call "R," we find depends somewhat upon the size and fineness modulus of the coarse aggregate, the volume of voids in the rodded coarse aggregate, and largely upon the moisture content of the fine aggregate, and the proportions of fine and coarse aggregates used. These various relations are shown graphically in Fig. 1.

If we have 100 cu. ft. of stone, measured loose and rod it, the volume will shrink to 85 or 90 cu. ft., depending upon its size, the fineness modulus, and whether it is pebbles, crushed boulders or crushed limestone. The rodded volume, for various amounts of stone used in the mixture, is represented by the line "A-B." In this rodded volume there is a certain volume of voids, amounting to about 38 per cent in crushed boulders, and about 40 per cent in crushed limestone. The volume of voids is shown by the shaded area, or line "A-C."

If we measure, while in a loose condition, 100 cu. ft. of sand, dry it and rod it, the resulting volume of dry, rodded sand would vary from 74 to 90 cu. ft., depending upon the amount of moisture it originally contained. (This will be referred to later in Fig. 2.) If the sand were comparatively dry, the line "E-D" would represent the dry, rodded volume for various percentages of sand used. If it had contained more moisture, the line "E^1-D" would represent this volume.

Theoretically, until there is used in the mixture enough sand to fill the voids in the stone, the sand would not affect the dry, rodded volume of the mixture of the two. As the amount of sand is increased, and the voids are filled, the excess volume of sand increases the volume of the mixture. The intersection at "G" of the line "E-D" with the line "A-C" is the point where, theoretically, the sand would just fill the voids.

If we had started say with 60 ft. (60 per cent) of stone, from the diagram we find that the rodded volume would be 53 cu. ft., of which about 20 cu. ft. would consist of voids. Using with the stone 40 ft. of dry sand—(40 per cent)—the volume of sand, when dried and rodded, would be 36 cu. ft., of which 20 cu. ft. would be required to fill the voids in the stone. Then for this particular mixture of sand and stone, the theoretical mixed, dry, rodded volume would be (Stone) 53 + Sand (36-20) or 69 ft., or, having started with a total of 100 cu. ft. of sand and stone, the ratio of True to Nominal Volume would be 69 per cent. In a like manner the line

E-F-B can be plotted for any percentage of sand and stone used, and it represents the theoretical ratio of the True Volume to the Nominal Volume, or True Mix to Nominal Mix. If the sand contained more moisture, this ratio would be represented by the line E^1-F^1-B.

The tests which were made, fit the wings of these curves nicely, but would not follow into the sharp elbow at F, but fell along a line such as E-H-B. This is due to the fact that it is impossible to completely fill the

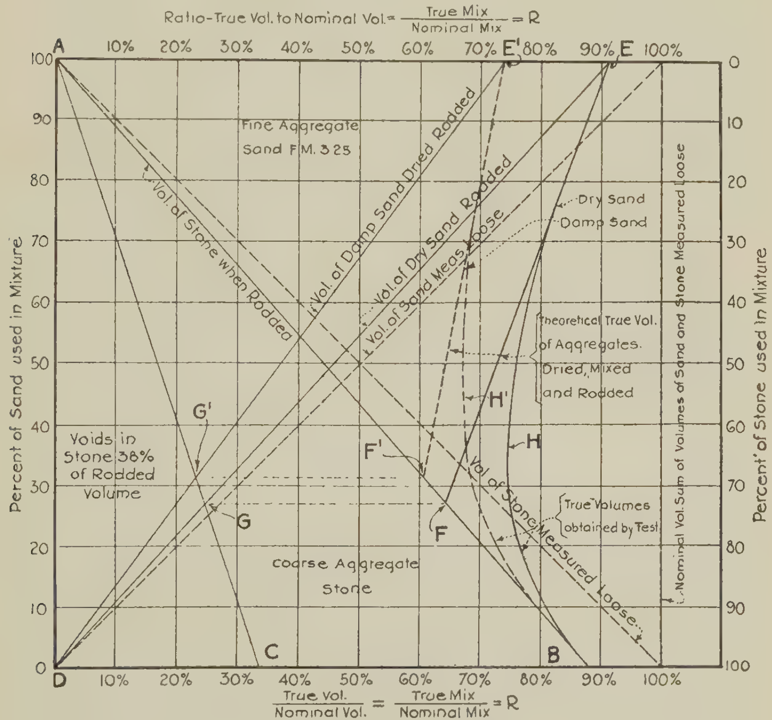


FIG. 1.—RELATION OF VOLUMES OF DRIED AGGREGATES MIXED AND RODDED TO SUM OF VOLUMES OF AGGREGATE MEASURED LOOSE.

voids in the stone, with the sand, and that in mixing the sand with the stone, it causes a separation of the particles of stone and increases the stone volume.

If it were possible to fill every void in the stone with the dry sand, the point F should be obtained, but, assuming that in preparing the dry, mixed and rodded aggregate in the laboratory, the voids at this critical point were no more nearly filled than we could fill them in our tests, the curve E-H-B, or E^1-H^1-B, for wet sand more nearly represents the proper value of R to be used.

The variation in the F. M. of the stone has but little effect on the volume when rammed. We have found that crushed boulders, having a F. M. of 8.0, when rodded give a volume of about 89 per cent of the original; with a F. M. of 7.5, a rodded volume of about 88 per cent, and a F. M. of 7.0, a rodded volume of about 87 per cent. Crushed limestone, having a F. M. of 8.0, gives a rodded volume of about 90 per cent; a F. M. of 7.5 a rodded volume of about 89 per cent, and a F. M. of 7.0 a rodded volume of 88 per cent. It can be seen from Fig. 1, that this small variation in rodded volumes of the coarse aggregate would change but little the line E-H-B, or E^1-H^1-B.

For our own work the fineness modulus of the available sands remains fairly constant and has an average value of about 3.3, but, as in all outside work, the amount of moisture they contain varies continuously. The moisture, within certain limits, causes the sand to bulk or increase in volume, and consequently, when mixed with stone, affects the ratio "R."

On Chart A, Fig. 2, is plotted the results of tests of dry, rodded volumes obtained from known volumes of loose sand with a moisture content varying from 0 to 30 per cent by weight of the dry sand. It will be seen that for dry sand, the dry, rodded volume is 91.5 per cent of the loose volume. The greatest shrinkage due to drying and rodding is with a moisture content of about 7 per cent, when the theoretical value of "R" would apparently be about 71 per cent, although 73.7 per cent was the minimum obtained in the tests. With a further increase in the water content the sand really becomes flooded, the bulking action is replaced by a puddling action, and the value "R" again increases, and when we have about 28 per cent moisture, the material has become so packed that there is no greater shrinkage in volume, when it is dried and rodded, than occurs with dry sand. In outside stock piles we find a minimum of about 2 per cent moisture, and a maximum of 8 per cent or 10 per cent, after heavy and continued rains.

On Chart B, Fig. 2, is shown the relation between the weights per cu. ft. of sands containing various percentages of moisture, and the moisture content. It is interesting to note, that for this particular sand, the weight per cu. ft. of the wet sand, with moisture content of P per cent, is $(110 + P)$ times the ratio (R) of the volume of dry, rodded sand to the original volume of loose, wet sand. By this chart we can determine closely the moisture content, by simply weighing a cu. ft. of the loose, wet sand.

In Chart C, Fig. 2, the weight of the wet sand is plotted against the ratio R.

Fig. 3 is the chart we are now using to design our concrete mixes, that is, to determine our Nominal Mix, when our True Mix is known. The values of R for 100 per cent sand (top of chart) are taken from Chart A, Fig. 2. Curves for sand containing more than 10 per cent of moistures are not plotted, as we do not have to work with them. They would fall between the extremes shown, as may be seen from Chart A, Fig. 2. That is,

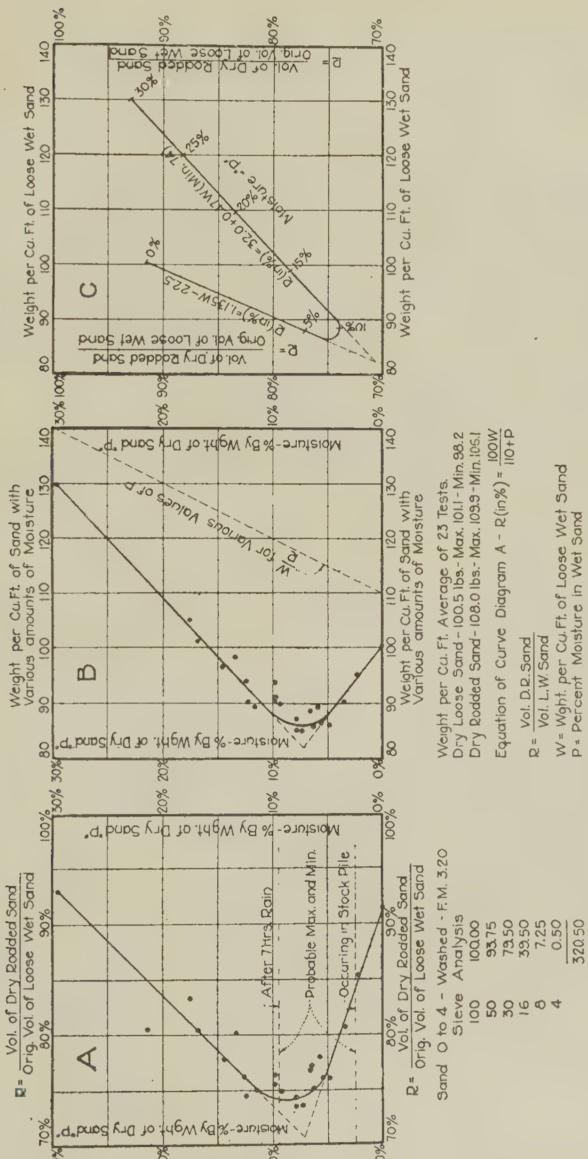


FIG. 2.—RELATION OF VOLUME OF LOOSE WET SAND TO VOLUME OF DRY RODDED SAND.

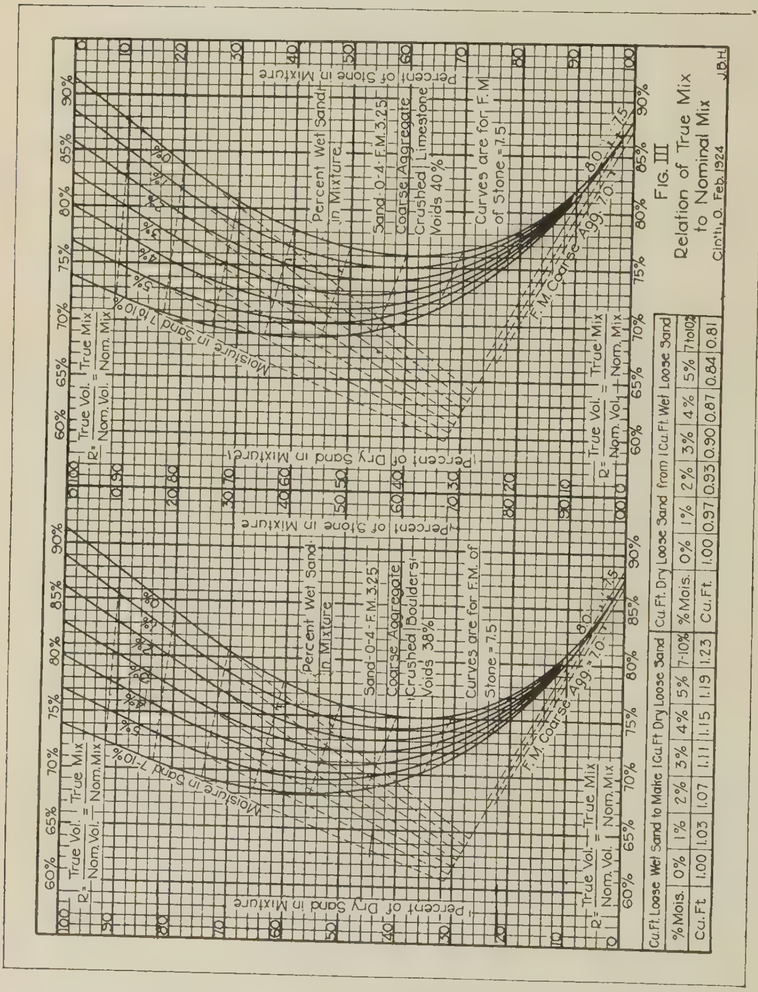


FIG. 3.—RELATION OF TRUE MIX TO NORMAL MIX

the value R, for a sand having 15 per cent moisture, would be the same as that for a sand having about 4½ per cent moisture.

This chart (Fig. 3) clearly shows that the amount of moisture in the sand affects the ratio of True Volume to Nominal Volume more than any other characteristic of the aggregates. The percentage of sand used, in the mixture will usually fall between 30 and 50 per cent, and within this range, the value R varies from 67 to 76 per cent.

It is not claimed that this diagram is perfect, but its development seems to be theoretically sound, and it has been fairly well confirmed by tests. We will continue to use it, until we find something better, and feel that the resultant error will probably be less than that due to the inaccurate measurement of the batches. It is based on materials which we are using and would perhaps have to be modified for other aggregates, especially for different sands.

As an example of its use, we will assume that to obtain the proper fineness modulus, with dry materials, we should use 40 per cent of sand and 60 per cent of crushed boulders, and that with a certain consistency, the required True Mix for the desired strength is 1: 3.6. From Fig. 3 we find that for dry materials the value of R to be 0.75, and the Nominal Volume of loose, dry, separate aggregates is then $3.6 \div 0.75 = 4.80$ cu. ft., and the Nominal Mix is 1: 4.8. Of this 4.8 cu. ft., 40 per cent or 1.92 cu. ft. is dry sand and 2.88 cu. ft. is stone; or the proportions for the batch are 1: 1.92: 2.88.

However, we find that the sand we are using contains 2 per cent moisture, which causes it to swell or bulk, resulting in a change in the factor R, the Nominal Mix and Proportions. To correct for the various moisture contents of the sand, the diagonal dotted lines have been drawn in Fig. 3. From them we can find the value of the factor R to give the correct True Mix, and the per cent of moist sand to be used to produce the desired Fineness Modulus. In this example, begin at the intersection of the dry sand curve with the horizontal line representing 40 per cent sand, and follow the dotted diagonal line upward to the left to its intersection with the 2 per cent moisture curve, and find the value of R to be 73 per cent, and the percentage of sand 41.7 per cent. The Nominal Volume, with sand containing 2 per cent moisture, becomes $3.6 \div .73 = 4.93$ cu. ft. and the Nominal Mix is 1: 4.93. Of the Nominal Volume, 41.7 per cent or 2.05 cu. ft. is sand, and the proportions are, 1: 2.05: 2.88.

After a heavy rain the sand might contain 8 per cent of moisture. Continuing along the same dotted diagonal line to its intersection with the 8 per cent moisture curve, we find R to be 69 per cent and the percentage of sand 45 per cent. The new Nominal Volume becomes $3.6 \div 0.69 = 5.22$ cu. ft. Of this volume, 45 per cent or 2.34 cu. ft. is sand and 2.88 cu. ft. is stone, and the proportions are, 1: 2.34: 2.88.

The quantity of loose stone used in the mixture remains constant, as moisture does not affect its volume. Moisture in sand, as ordinarily

encountered, causes the grains to separate, or the sand to bulk, and to obtain the same number of grains of sand that we would have in a given volume of dry sand, we must increase the measured volume of the loose damp sand in making up the batch, as the Nominal Mix and proportion of sand varies with moisture content of the sand.

The proportions, 1: 1.92: 2.88 for dry sand; 1: 2.05: 2.88 for sand containing 2 per cent moisture; and 1: 2.34: 2.88 for sand containing 8 per cent moisture, are all equivalent to the same True Mix of 1: 3.6, and the same Fineness Modulus of the combined aggregates.

Where it is impractical to change the amount of sand used in the batch, with frequent changes in the moisture content, an average of 3 per cent or 4 per cent moisture might be assumed, in porportioning the batch.

In order that our engineers and inspectors may more easily become acquainted with the methods of designing and making concrete mixtures, a set of instructions has been prepared, explaining as simply as possible the effect of variation in quantity and quality of the ingredients, with typical examples of mixture design worked out for them. They are reprinted in an appendix and offered merely as information.

APPENDIX

NOTES ON DESIGNING CONCRETE MIXTURES AND INSTRUCTIONS TO MASONRY INSPECTORS.

Concrete is made of cement, sand, stone and water, and any material change in the quantity or character of any ingredient will affect the strength and quality of the finished concrete.

Knowing the quantity and character of the various ingredients, if properly mixed and placed, the strength or quality of the concrete can be determined; or conversely, if a certain strength of concrete is desired, this strength can be closely obtained by combining the proper ingredients in the right proportions.

It having been established that concrete of a pre-determined strength can be made with careful selection and proportioning of materials and proper mixing and placing, it is evident that concrete so made is economical, as we have neither excess nor insufficient strength for the requirements of the structure, and usually the concrete will be better and more uniform in quality than that mixed under the usual arbitrary proportions.

AGGREGATES.

Aggregates of the proper size and grading, and the combining of the sand and stone in the right proportions materially increase the strength of concrete, and assuming a given consistency and mix, the strength depends on the size and grading of the aggregate. Generally the larger and coarser the aggregate, the stronger the concrete. Coarse sand and the larger size pebbles produce greater strengths than the finer sands and smaller stones.

The term "Fineness Modulus" (hereinafter written F. M.) has been developed to indicate the size and grading of aggregates and their value for use in concrete. It is applicable to either fine or coarse aggregates or any combination of them.

The F. M. of an aggregate is determined by a sieve analysis, made with a set of U. S. Standard Square Mesh Sieves, each sieve having clear openings double the width of the next smaller size. The sizes are 100, 50, 30, 16, 8 and 4 meshes to an inch, and ⅜ in., ¾ in. and 1½ in. The percentage of aggregate, (preferably by weight) coarser than each sieve is determined. The sum of these percentages divided by 100, is called the Fineness Modulus. After the F. M. of the fine and coarse aggregates have been

determined, the F. M. of a mixture of the aggregates in any proportion can be found, or the proportion of fine and coarse aggregates to produce a certain F. M. in the mix. A sample sieve analysis of various aggregates is shown below:

	Per Cent Coarser than each Sieve.									Fineness Modulus.
Sieve	100	50	30	16	8	4	⅜	¾	1½	
Sand 0 to ¼ in.	100	95	81	42	6	1	0	0	0	3.25
Pebbles ⅜ to 1½ in.	100	100	100	100	100	98	96	81	6	7.81
Limestone ⅜ to 1 in.	100	100	100	100	100	98	88	28	0	7.14
Sand and pebbles 0 to 1½ in.	100	98	95	87	77	61	44	30	7	5.99

There may be a wide variation in the results of the sieve analysis of different aggregates, but as long as the F. M. remains the same, uniform results may be expected in concrete, and by combining the sand and stone in the proper proportions the F. M. of the mixture can be made that which is best suited to the purpose.

As the maximum size of the aggregate increases, the F. M. increases and, until it becomes too coarse for the amount of cement used, the strength of the concrete increases. Larger F. M. can be used with rich mixes than with lean ones, and with round pebbles than with crushed limestone. Therefore, the best value of the F. M. will depend on both the mix and the maximum size and character of the aggregate. Larger values can be used in mass work than in light reinforced work.

When the F. M. of the sand and stone is known, the F. M. of a mixture of the two, in any proportion. can be obtained by the Formula

$$M = \frac{r\,M_f + (100\text{-}r)\,M_c}{100} \qquad (1)$$

Where M = F. M. of Mixture.
M_c = F. M. of Coarse Aggregate.
M_f = F. M. of Fine Aggregate.
r = Percentage of Fine Aggregate Used in the Mixture.

Or, if a certain F. M. of the mixture is required, the percentage of fine aggregate to be used can be determined by the Formula

$$r = \frac{M_c - M}{M_c - M_f} \qquad (2)$$

The desirable values of the F. M. of the sand (for mortars) and for the mixed aggregates (for concrete) for various mixes and maximum sizes of aggregates are shown in the following Tables I and I-a:

TABLE I.—DESIRABLE VALUES OF FINENESS MODULUS FOR LIGHT REINFORCED WORK.

True Mix, Volume of Cement to Mixed Aggregate.	For Mortar.		For Concrete.					
	Sand.		Sand and Pebbles.			Sand and Crushed Stone.		
	0–4	0–⅜ in.	0–¾ in.	0–1 in.	0–1½ in.	0–¾ in.	0–1 in.	0–1½ in.
1 to 1.............	4.75	5.60	6.50	6.90	7.35	6.25	6.65	7.10
1 to 2.............	4.20	5.05	5.90	6.30	6.70	5.65	6.05	6.45
1 to 3.............	3.90	4.70	5.50	5.90	6.30	5.25	5.65	6.05
1 to 4.............	3.60	4.40	5.20	5.60	6.00	4.95	5.35	5.75
1 to 5.............	3.45	4.20	5.00	5.40	5.80	4.75	5.15	5.55
1 to 6.............	3.30	4.05	4.85	5.25	5.65	4.60	5.00	5.40
1 to 7.............	3.20	3.95	4.75	5.15	5.55	4.50	4.90	5.30

TABLE Ia.—DESIRABLE VALUES OF FINENESS MODULUS FOR MASS WORK

True Mix, Volume of Cement to Mixed Aggregate.	Concrete.			
	Sand and Pebbles.		Sand and Crushed Stone.	
	0–1 in.	0–1½ in.	0–1 in.	0–1½ in.
1 to 1..	7.00	7.55	6.75	7.30
1 to 2..	6.40	6.90	6.15	6.65
1 to 3..	6.00	6.50	5.75	6.25
1 to 4..	5.70	6.20	5.45	5.95
1 to 5..	5.50	6.00	5.25	5.75
1 to 6..	5.35	5.85	5.10	5.60
1 to 7..	5.25	5.75	5.00	5.50

Fine aggregates used in concrete work should not have a higher F. M. than that shown for Sand 0-4, for Mortars, of the same mix. While the above values will give maximum strengths they may occasionally have to be reduced, to get a concrete which will work properly, particularly if the coarse aggregate contains a large percentage of coarse particles.

MIX.

The strength of concrete, assuming the size and grading of the aggregate and consistency to remain constant, depends upon the mix. This mix, as used in the charts and tables, refers to the *TRUE MIX*, and represents the volume of mixed, dry and rodded aggregate used with one volume of cement.

The use of "True Mix" is convenient for laboratory work, but such a condition of aggregates is not encountered in the field. As ordinarily used, the aggregates, especially the sand, contain more or less water, depending upon weather conditions, and are always loose or bulked.

To eliminate these variables the mixes for certain strength of finished concrete must be expressed in terms of True Mix, but in the manufacture

of concrete we must measure the sand and stone separately, not rodded or tamped and containing moisture, so that, in order to have concrete of the required True Mix, we must know what volumes of loose, damp, sand and stone, which may be called the "Nominal Volume," will be required to make a certain volume of mixed aggregates dry, rodded, which may be called the "True Volume." The ratio of the True Volume to the Nominal Volume depends upon the F. M. of the aggregates, the moisture content of the sand, the voids in the coarse aggregate, and the proportions of fine and coarse aggregates used, and is shown in Diagram II. It will be seen that for the usual mixtures this factor varies from about 65 per cent to 75 per cent. That is, 100 cu. ft. (the sum of the separate volumes) of sand and stone, measured when loose and damp, would, if dried, mixed, and rodded, produce 65 to 75 cu. ft. of dried rodded aggregate, depending upon the size of the material, moisture content of the sand, and proportions of sand and stone used.

This factor permits us to determine what Nominal Volume will produce a certain True Volume, or what Nominal Mix would be required to give a certain True Mix. The NOMINAL MIX being the sum of the volumes of the separate aggregates, measured loosely in their normal condition, used to one volume of cement. For example, in a mixture of 1 part cement, 2.5 parts sand and 4.5 parts stone, the Nominal Mix would be 1 to $(2.5 + 4.5) = 1$ to 7. If the proper factor from Diagram II were 70 per cent, the True Mix would be 1 to $(7. \times .70)$ or 1 to 4.9.

Usually separate aggregates will be furnished, consisting of washed sand, 0 to ¼ in. in size, and pebbles and crushed boulders or crushed limestone, ⅜ to ¾ in., ⅜ to 1 in., ⅜ to 1½ in. in size, depending upon the character of the work. The proportions of sand and stone used will depend upon the desired F. M. of the mixture. Ordinarily the amount of sand used will be between 20 per cent and 60 per cent, by volume, of the mixture.

When sand is mixed with stone, the volume of mixed aggregate is less than the sum of the volumes of the separate aggregates, and under ordinary conditions this ratio is between 80 per cent and 90 per cent, depending upon the moisture content, size of aggregate and proportion of sand and stone.

In the preceding example the 2.5 cu. ft. of loose sand added to 4.5 cu. ft. of loose stone, would produce say—6.0 cu. ft. of loose mixed aggregate. If one sack of cement were used with the sand and stone as before, the APPARENT MIX, as it may be called, would be 1 to 6.0. The term "APPARENT MIX," however, would not be used except when a premixed aggregate (concrete gravel) is used.

If the 6.0 cu. ft. of mixed aggregate were dried and rodded or tamped it would shrink (using the same True Mix factor as before) to 4.9 cu. ft.

For certain unimportant work a premixed aggregate (concrete gravel) may be furnished. This is either run of pit gravel as it comes from the bank, or gravel washed and graded to some extent to produce the proper proportions of sand and pebbles. Its use should be discouraged, as its F. M.

will be found to vary continuously over wide ranges and consequently it is difficult to maintain a uniform and correct mixture.

Where it is used the same F. M. as used for separate aggregates, sand and pebbles, should be maintained, by adding sand or pebbles as may be necessary. The volume of premixed aggregates used to one volume of cement is the Apparent Mix. The ratio between the True Mix and Apparent Mix can be assumed as 85 per cent.

Aggregates should be sound structurally, free from organic impurities, and clean, without coatings and dust. Inasmuch as sand and pebbles are usually furnished washed, and from tested pits, they are likely to be satisfactory. They should be examined, however, and any dirty materials rejected. Crushed limestone, especially should be watched, as this often comes full of dust and screenings. Organic impurities in the sand can be identified by the Colorimetric Test, described in detail in the attached leaflet.

Frequent sieve analyses should be made of the fine and coarse aggregates, and should any marked variation occur in either or both of them, the proportions of sand and stone should be varied (Formula 2) so as to give the required F. M. of the mixture of the two.

In proportioning the batches care should be exercised to see that the amounts of sand and stone are measured with accuracy, so as to get a uniform mix. Due to the change in the moisture content of the sand, the Nominal Mix may have to be changed to maintain the same True Mix. When the mixed is charged from hoppers, or cars, lines or points can be marked for certain volumes, and the aggregates leveled off to these lines. If wheelbarrows are used, they can be marked for certain volumes, and the materials in them leveled off by means of a templet. With hopper scales the aggregates can be accurately proportioned by weight, after the weights per unit volume of the aggregates are known.

MIXING WATER.

The amount of water used in mixing concrete affects its strength as much as the quantity of cement used. It has two functions; to hydrate the cement, and to lubricate or make the aggregate workable. The strength of the concrete depends upon the "Water Cement Ratio" or ratio of volume of mixing water to the volume of cement. The smaller this ratio, the stronger the concrete, as long as there is enough water to make it workable. Decreasing the amount of water increases its strength. A dry mix, but still workable, will, with the same proportions of cement and aggregate, be about twice as strong as if water were added to produce a sloppy mix. This means that to secure a certain strength, if a dry mix instead of a very wet one is used, only about one-half the cement would be required. As an example, to produce a concrete having a strength of 2500 lb., if it were mixed wet, would require a True Mix of about 1: 2.2; if it were mixed dry, but still be workable, the True Mix would be about 1-4.6.

Rich mixtures require less mixing water per sack of cement than lean ones, as there is a smaller volume of aggregate per sack of cement; coarse aggregates require less mixing water for the same consistency than fine aggregates. Then the quantity of mixing water may be changed, due to

(1) Change in Mix (cement content).
(2) Change in Size and Grading of Aggregate.
(3) Change in Consistency.

The required amount of mixing water, includes the water which is contained in the aggregate as well as that added at the mixed. The total amount of mixing water required for various mixes, aggregates, and consistencies is known, but this particular amount includes that in the aggregate which is generally unknown and a variable, so that the amount to be added to the batch to give certain consistencies is a variable.

A simple method of determining and regulating the consistency of concrete is by the slump test. The apparatus consists of a truncated cone, made of sheet metal, with a handle or grip on either side. The frustrum of the cone has a diameter of 4 in. at the top, 8 in. at the bottom, and is 12 in. high.

The form is placed on a level surface and is filled, with the concrete to be tested, in layers about 4 in. deep, each layer being rodded 30 times with a pointed rod; the top being leveled off. Immediately after the form is filled, it is graduaally lifted, vertically, and the amount the concrete settles or slumps, represents the slump.

Concrete having a slump of ½ to 1 in. will contain but little more water than that necessary to produce the maximum strength, but would be a little too dry to use in construction work. Concrete having such a slump is said to have a "Relative Consistency," for ordinary mixes, of 1.00. Concrete containing 10 per cent more water has a Relative Consistency of 1.10 and would have a slump of 3 to 4 in. Concrete containing 25 per cent more water (Relative Consistency 1.25) would have a slump of 6 to 7 in., and 50 per cent more water (Relative Consistency 1.50) would have a slump of 8 to 10 in.

To again emphasize the importance of using as little water as possible, the expected strengths for various slumps, or Relative Consistency, with the same aggregate and mix is shown below:

Slump	Rel. Cons.	F. M.	True Mix	Strength
½-1 in.	1.00	5.8	1 to 4	3250
3-4 "	1.10	5.8	1 to 4	2850
6-7 "	1.25	5.8	1 to 4	2200
8-10 "	1.50	5.8	1 to 4	1500

It will ordinarily be possible to work a concrete having a slump of 3 to 4 in., but if but little more water is added so that a slump of 8 to 10 in. is obtained, and much concrete is poured this wet, the strength will be only half of the drier concrete using the same amount of cement and aggre-

gate. If more water is used it is necessary to increase the amount of cement to secure the same strength, and this, of course, is wasteful. *There should always be used the smallest amount of mixing water possible to give a mixture sufficiently plastic for the work at hand.*

The inspector must make frequent slump tests, and insist that the control of the amount of water used be such as to insure a uniform and the desired slump. The water should come to the mixer from a container, which can be graduated, or otherwise control the amount of water added at the mixer.

For the same mix and the same relative consistency, coarse aggregates require less water than fine ones, and the amount of water required for the same relative consistency decreases somewhat as the time of mixing increases.

Consistencies of concrete, as represented by the slump, which should be used for various classes of work are as follows:

TABLE II.

Type of Construction.	Maximum Slump, in.
Mass Concrete	3 to 4
Reinforced Concrete:	
Heavy sections, reinforcement more than 4-in. centers	3 to 4
Heavy sections, reinforcement less than 4-in. centers	4 to 5
Light sections, but easily tamped, reinforcement more than 4-in. centers	3 to 4
Light sections, or thin vertical walls, which are difficult to tamp	5 to 6
Floors, in buildings, platforms and pavements	1½ to 2½

The above slumps represent the maximum which should be permitted, and if a drier concrete can be worked satisfactorily, as determined by trial, lower slumps should be used.

The mixing water used should be clean, containing but little, if any, sediment, and free from organic matter, acids and alkaline salts.

CEMENT.

The cement is usually furnished by the railway company and is inspected at the mill, and the cars sealed, so that it can usually be depended upon as meeting the specifications. If the inspector notices, however, anything unusual as to its condition, its time of setting or other qualities, he should at once notify the engineer in charge. The cement should be stored in such a manner that it can be kept absolutely dry, and the storage should be as nearly air tight as possible. Any damp set cement should be discarded.

The size of the batch is usually determined by the capacity of the mixer, the total volume of aggregates remaining constant, so that after the Nominal Mix is found from the required True Mix, the amount of cement required per batch is frequently found to be so many bags and a fraction. It is difficult to measure the cement nearer than to the half bag, and if,

for instance, it were found that 4¼ bags were required to the batch, 4½ bags would be used, or 4½ bags might be used in one batch and 4 bags the next, averaging 4¼ bags.

The inspector should keep a daily count on the bags of cement used, batches mixed, and the amount of concrete placed, so as to check the Nominal Mix and yield. If the mix is changed during any day's run, the count and measurement of concrete placed should begin again at the time of changing the mix.

Quantities of Materials Required for Making Concrete.

The volume of concrete obtained from a unit volume of aggregate is affected somewhat by the size and fineness modulus of the aggregate used, and the Relative Consistency of the mixture, and largely by the Mix. This

TABLE III.

True Mix.	Volume of Dry Rodded Aggregate.	Volume of Concrete Produced.					
		Aggregate 0 to ¾ in.			Aggregate 0 to 1½ in.		
		Relative Consistency and Slump.			Relative Consistency and Slump.		
		1.00 ½–1 in.	1.10 3–4 in.	1.25 6–7 in.	1.00 ½–1 in.	1.10 3–4 in.	1.25 6–7 in.
1 to 7........	100	101.5	102.0	103.0	99.0	99.5	100.5
1 to 6........	100	102.5	103.5	104.5	100.0	101.0	102.0
1 to 5.5......	100	103.5	104.5	106.0	101.0	102.0	103.5
1 to 5.0......	100	105.0	106.0	107.5	102.5	103.5	105.0
1 to 4.5......	100	107.0	108.0	110.0	104.0	105.0	107.0
1 to 4.0......	100	109.0	110.0	112.0	106.5	107.5	109.5
1 to 3.5......	100	112.0	113.0	114.5	109.5	110.5	112.0
1 to 3.0......	100	116.0	117.0	119.0	113.0	114.5	116.5
1 to 2.5......	100	120.5	122.0	123.5	118.0	119.5	121.0

ratio is known as the yield and can be conveniently expressed in terms of the volume of dry rodded aggregate (True Volume), and the True Mix. From Diagram II the ratio between the volumes of dry rodded aggregate and separate loose aggregates can be found, and by applying the yield factor corresponding to the True Mix used the total volume of sand and stone to produce a certain volume of concrete can be determined. The proportion of sand and stone needed is known after the per cent of sand "f" is found. The yields are shown in the following Table III. They are based on a F. M. of about 5.8. For a F. M. of 5.6 they should be increased about 1 per cent; for a F. M. of 6.0 they should be decreased 1 per cent; and for a F. M. of 6.2 decreased about 2 per cent. No allowance for waste is included.

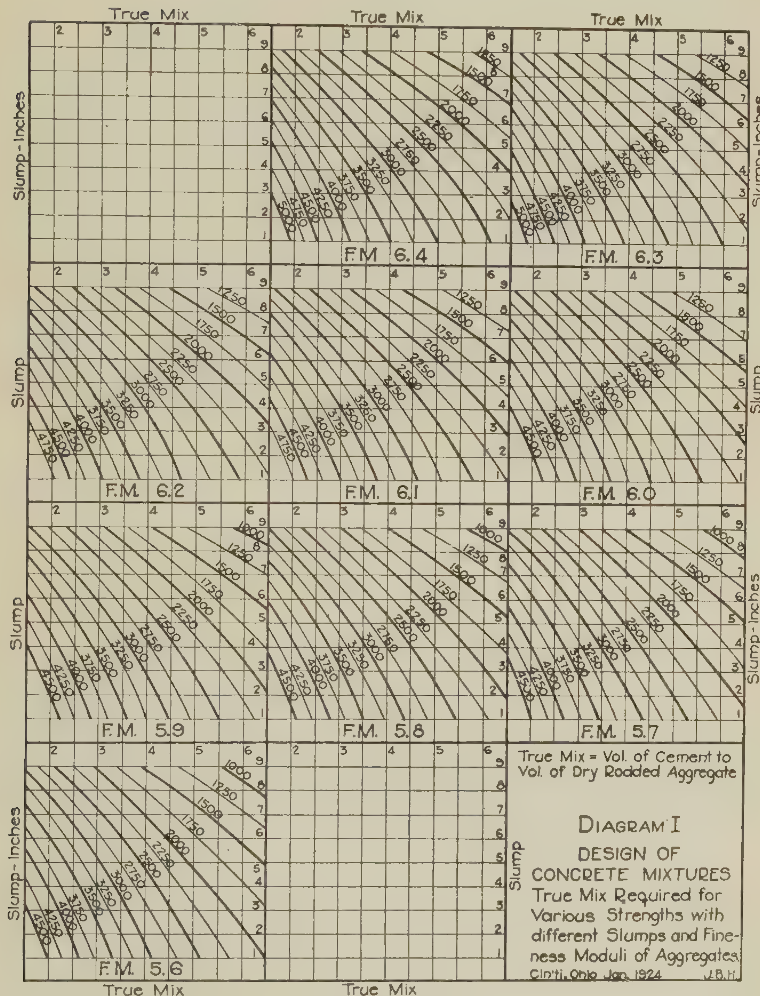


DIAGRAM I.—DESIGN OF CONCRETE MIXTURES.

EXAMPLE OF A DESIGN OF CONCRETE MIXTURE.

Suppose that we are given a concrete arch to build, the plans for which call for concrete in the abutments to have a strength of 2000 lb. and in the arch ring of 3000 lb. per sq. in.

The aggregates furnished are sand, 0 to ¼ in., and crushed boulders, ⅜ to 1½ in. We will assume, for trial purposes, for the 2000 lb. concrete, a F. M. of 5.8 and for the 3000 lb. concrete a F. M. of 6.0. The work is of such character that from Table II we find the slump to be used is 3 to 4 in. To be on the safe side we will use, to design the mix, a 4 in. slump.

Referring to Diagram I, we find that to obtain 2000 lb. concrete, using a F. M. of 5.8 and a slump of 4 in. will require a True Mix of 1: 5.7; and 3000 lb. concrete—F. M.—6.0, Slump 4 in.—a True Mix of 1: 3.8. For these approximate mixes we find, from Table I-a, that the correct F. M. would be 5.9 and 6.2. Referring again to Diagram I, and using a F. M. of 5.9, we find for the 2000 lb. concrete, the True Mix is 1: 5.8 and with a F. M. of 6.2 for the 3000 lb. concrete, the True Mix is 1: 3.9. That is, if we use one sack of cement with 3.9 cu. ft. of the dried and rodded aggregate, we can expect a concrete having a strength of 3000 lb. per sq. in.

Suppose the sieve analysis shows that the sand has a F. M. of 3.25 (M_f) and the stone a F. M. of 7.75 (M_c). Then from Formula 2:

For 2000 lb. Concrete

$$M = 5.9 \qquad r = \frac{7.75-5.9}{7.75-3.25} = 42\% \text{ Sand when dry.}$$

For 3000 lb. Concrete

$$M = 6.2 \qquad r = \frac{7.75-6.2}{7.75-3.25} = 35.2\% \text{ Sand when dry.}$$

If the sand used were dry, we find from Diagram II (r being 42 per cent for the 2000 lb. concrete) the ratio of True Volume to Nominal Volume is 0.75. Then to produce 5.8 cu. ft. of dry rodded aggregate would require 5.8 ÷ 0.75, or 7.60 cu. ft of sand and stone, giving a Nominal Mix of 1: 7.6. Of this, 42 per cent or 3.19 cu. ft. will be sand and 4.41 cu. ft. stone, and the proportions are 1: 3.19: 4.41. If the sand contained, say, 5 per cent moisture it would require, (See table on Diagram II) 1.19 cu. ft. of such loose wet sand to make 1 cu. ft. of loose dry sand, and we would therefore have to use in the batch 3.19 x 1.19 or 3.80 cu. ft. of this wet sand, with 4.41 cu. ft. of stone, and the proportions become 1: 3.80: 4.41. The Nominal Mix is 1: 8.21, and the percentage of sand used is 3.80 ÷ 8.21 = 46.3 per cent.

The Nominal Mix, for sand containing any amount of moisture, can be also read directly from Diagram II. Starting at 42 per cent on the curve for dry sand, and following the diagonal line up to the left to the point of intersection with the 5 per cent moisture curve, we find the factor

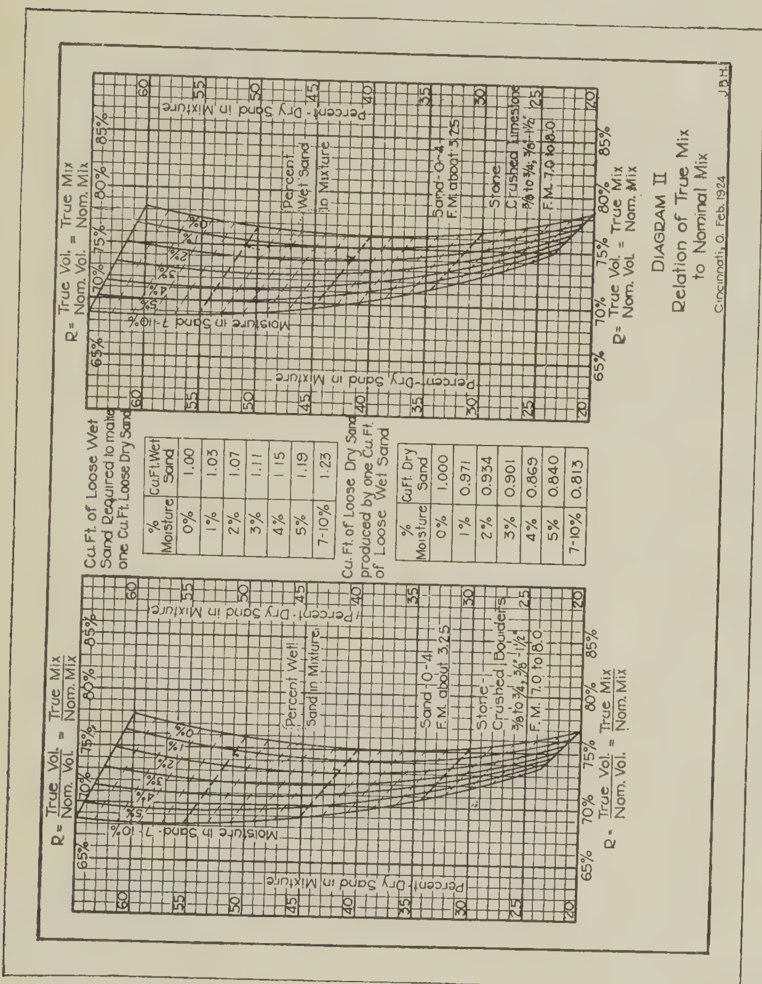


DIAGRAM II.—RELATION OF TRUE MIX TO NOMINAL MIX.

"R" to be 70 per cent and percentage of sand 46.5 per cent. Then the Nominal Volume is $5.8 \div .70 = 8.28$, of which 46.5 per cent is sand, 3.85 cu. ft., and 4.43 cu. ft. stone, or practically what we get by computation.

In the same manner we find for the 3000 lb. concrete, True Mix 1: 3.9, $r = 35.2$ per cent, that the factor for dry sand is 0.75; the Nominal Mix is 1: 5.20, and the proportions are 1: 1.83: 3.37. With sand containing 5 per cent moisture the factor R is 0.705 and the Nominal Mix is 1: 5.53, and the proportions are 1: 2.16: 3.37.

The variation in the Nominal Volume, and proportions, due to change in the moisture content of the sand, is clearly shown by the above example. In dry weather the sand will rarely contain less than 2 per cent moisture, and even after heavy rains, more than 8 per cent, but the amount of moisture in it should be determined daily. This can be done by taking a small volume, not less than 0.1 cu. ft., and weighing it while wet. Dry it and weigh, the difference in weight being the weight of water. The weight of water divided by the weight of dry sand will give the percentage of moisture originally contained.

Facilities are not always available for drying the sand, and a close approximation can be had as to the moisture content by determining the weight per cu. ft. of the wet sand. This weight can be found by filling a pail of known volume with the loose damp sand and weighing it. From Diagram III, the per cent of moisture can be found when the weight per cubic foot is known. This method is so simple, that the inspector can make the test several times daily, if necessary.

It is not always practicable to change the amount of sand used in the batch, with each change in the moisture content, but it should be done whenever possible, or when the change is a material one. In establishing the batch it would be well to assume 2 per cent moisture in the sand, and when it is wetter, increase the volume of sand used if practicable. If the volume is not increased no particular harm is done, except that the mixture will be a little coarser or harsher, but this is not objectionable as long as it works well.

If the F. M. of the fine or coarse aggregates, or both should change, the new value of r should be computed, and the proportions varied accordingly.

Assuming that the mixer has a capacity of 30 cu. ft., we could use in each batch (5 per cent moisture in sand), for the 2000 lb. concrete, 3.5 sacks of cement, 13.3 cu. ft. of sand, and 15.4 cu. ft. of stone, and for the 3000 lb. concrete, 5 sacks of cement, 10.8 cu. ft. of sand, and 16.9 cu. ft. of stone.

We will now determine the materials required for the structure, assuming that there are 1200 cu. yd. of 2000 lb. concrete and 700 cu. yd. of 3000 lb. concrete.

For the 2000 lb. concrete the True Mix is 1: 5.8; the F. M., 5.9; slump 3 to 4 in.; aggregates 0 to 1½ in. From Table III, interpolating and correcting for the F. M. of 5.9, we find the yield to be 101 per cent,

or to produce 100 cu. yd. of finished concrete would require 100 ÷ 101 = 99 cu. yd. of dry rodded aggregate.

From Diagram II, using dry sand, we find to produce 99 cu. yd. of dry rodded aggregate would require 99 ÷ 0.75 or 132.0 cu. yd. of sand and stone, measured separately, when in a loose or bulked condition.

With a true mix of 1: 5.8 the amount of cement per 100 cu. yd. of finished concrete is (99 ÷ 5.8) 27 = 461 cu. ft. or 115.25 bbl.

Then for the 1200 cu. yd. of 2000 lb. concrete there would be required:

Cement	12 x 115.25	= 1383 bbl.
Dry Sand and Stone	12 x 132.0	= 1584 cu. yd.
Dry Sand—42 per cent or	1584 x .42	= 665 cu. yd.
Sand as received (4 per cent moisture)	665 x 1.15	= 765 cu. yd.
Stone—58 per cent		= 919 cu. yd.

For the 3000 lb. concrete we find, in the same way, that 100 cu. yd. of concrete (F. M., 6.2; True Mix 1: 3.9; Slump 3-4 in.) would require 100 ÷ 106 = 94.3 cu. yd. of dry rodded aggregate, or 94.3 ÷ 0.75 = 126.0 cu. yd. of sand and stone. And the amount of cement is (94.3 ÷ 3.9) 27 = 653 cu. ft. or 163.25 bbl. For 700 cu. yd. of such concrete would require:

Cement	7 x 163.25	= 1143 bbl.
Dry Sand and Stone	7 x 126.0	= 888 cu. yd.
Dry Sand—35 per cent	888 x .35	= 311 cu. yd.
Sand as received (4 per cent moisture)	311 x 1.15	= 358 cu. yd.
Stone—65 per cent		= 577 cu. yd.

The above quantities are net, and, for loss and waste, they should be increased about 2 per cent for cement and 5 per cent for the sand and stone.

MIXING AND PLACING.

Thorough mixing gives a better and stronger concrete. After all materials, including water, are in the mixer, the concrete should be mixed a minimum of one and one-half minutes and probably not to exceed three minutes. Concrete mixed two minutes is about 25 per cent stronger than that mixed only 15 seconds; but after that time it increases in strength so slowly as not to justify the additional time. Thoroughly mixed concrete is more plastic and more easily worked and handled, and will require somewhat less water to give equal slumps than when mixed for a shorter time.

The mixed concrete may be run directly from the mixer into the forms, or may be conveyed in buckets, wheelbarrows or buggies and dumped into the forms, or elevated to the top of a tower and distributed to the various portions of the work by chutes. This latter method has come into common use, particularly when the concrete to be placed is some distance from the central mixing plant, and it has probably been largely responsible for the wet mixes heretofore used. Rather than build a tower of sufficient height, lower towers with chutes having flat slopes are used, and the concrete is

made wetter so that it will flow. Sometimes the chutes are so steep that the coarse aggregate separates from the mortar, and rolls down into the forms. In fact, the use of this appliance has been so abused that it has come into general disfavor, and is not permitted on some work. It is often necessary to use it, however, particularly where the concrete must be distributed over a large area, from one central point, and if the aggregates are properly proportioned and the concrete thoroughly mixed, concrete can be successfully handled by chutes of proper design, without the use of excess water.

The chutes should be of metal or metal lined, and the different sections maintained at a uniform slope, which should not be flatter than 3 horizontal to 1 vertical, or steeper than 1½ horizontal to 1 vertical. A slope of the chutes, between these limits, should be used which gives the least segregation and clogging with the established consistency. If the Contractor desires to use a flatter slope, which may result in clogging, the inspector should insist that men be placed to keep the chutes clear, and not permit the use of additional water over that specified. It has been shown that comparatively dry mixtures, if properly made, run more freely along the chutes than the very wet mixes.

The end of the chute should be provided with baffle plates to keep the concrete from segregating as it drops into the forms, and where the end of the chute is more than 5 ft. above the surface of the concrete a spout must be used. The spout should be moved so as to deposit the concrete evenly over the area of the pit or form. When it is allowed to pile up the mortar runs out and it is impossible to get a concrete of uniform quality in the structure.

As the concrete is deposited in the forms, it should be well spaded or tamped, and the coarse material kept away from the forms to insure a smooth finish.

Finished concrete should be kept damp for two weeks, wherever possible, especially in summer, as thorough curing will increase its strength, and consequently its resistance to wear, about 50 per cent. This is particularly important in floors, and light re-inforced work, which dry out rapidly.

CONCRETE IN COLD WEATHER.

Frozen concrete is caused by the freezing of the mixing water before it can combine with the cement in the hardening process. If frozen concrete is thawed out and prevented from freezing again, it will harden properly. However, continued freezing and thawing will injure the concrete and may stop the hardening process altogether. Mass concrete can be placed in quite cold weather if certain precautions are taken to prevent freezing during the first few days, or until it hardens.

When the temperature is as low as 40 deg., both the aggregates and mixing water should be heated to a temperature of 175 to 200 deg. There is considerable heat generated in the concrete during hardening, and if this

heat is retained, there is but little danger of freezing except in thin sections. To retain this heat, the forms should be free from frost and ice. This can be accomplished with live steam, which will also warm the forms. The forms should be protected outside with canvas or straw for several days to prevent too rapid radiation. At the lower temperature it may be necessary to run steam lines between tarpaulin curtains and the forms.

Floors and pavements can be covered with straw or shavings. Anti-freezing compounds will not be used.

The warmer the concrete is kept the quicker it will harden, and, if possible, it should be kept at a temperature of about 60 deg. F. until it has hardened.

The forms cannot be removed as quickly in cold weather as in warm weather, and before allowing it to take any load it must be definitely determined that the concrete has hardened and is not frozen.

Responsibilities and General Duties of an Inspector.

1. It is the duty of the inspector to see that the work is executed in full accordance with the plans and specifications.

2. The inspector will be provided with complete plans and specifications for the work and shall familiarize himself with all of their requirements.

> Note.—The inspector should read and check off all dimensions and notes on the plans and list all special requirements in connection with the work to which they apply; when in doubt he should obtain instructions sufficiently in advance of the work to avoid delay.

3. The inspector shall familiarize himself with the Building Code or Regulations under which the work is being done, including all special rulings, which have a bearing on the type of construction under way.

> Note.—The inspector should be familiar with auxiliary specifications and the common tests for suitability of the materials used in concrete and re-inforced concrete work, such as:
>
> (a) Standard Specifications and Tests for Portland Cement.
> (b) Standard Specifications for Concrete Re-inforcement Bars, etc.
> (c) Standard Specifications for Concrete Aggregates.

4. The inspector shall be on the work at all times when it is in progress.

5. The inspector shall not permit work to proceed until requisite lines and levels have been established.

6. The inspector is authorized to:

(a) Forbid use of materials or workmanship which do not conform to specifications;

(b) Stop any work which is not being done in accordance with the plans and specifications;

(c) Require, with the approval of the engineer, the removal or repair of faulty construction.

7. The inspector shall report immediately to his immediate superior any action taken by him under Rule 6.

8. The inspector shall report at once to his immediate superior any discrepancies in plans or specifications that may come to his attention, and request instructions.

9. The inspector shall not permit any departure from plans or specifications without written instructions from his immediate superior.

10. The inspector shall record and report daily the following information:

(a) Date;
(b) Location of work under way;
(c) Name of contractor;
(d) Delivery, installation, and removal of contractor's equipment;
(e) Number and classification of laborers, teams, plant, etc., in use;
(f) Materials received and on hand;
(g) Inspection and location of forms erected and completed reinforcement in place;
(h) Location and approximate quantities of concrete placed and bags of cement used;
(i) Time of commencement and completion of placing concrete;
(j) Number and location of samples taken;
(k) Nature and time of any control tests made;
(l) Forms removed;
(m) All items on force account work;
(n) Weather conditions, and temperature readings taken morning, noon and evening, including also the maximum and minimum temperatures during each preceding period;
(o) Material rejected or work condemned under Rule 6;
(p) Disposition of rejected materials;
(q) Any special instructions given to contractor;
(r) Any unusual features of the day's work;
(s) Signature of inspector.

NOTE.—It is not expected that the inspector shall act as timekeeper or materials clerk; but will record the data called for and, so far as possible, indicate the source of the information. For convenience, the inspector should be supplied with printed forms for daily reports.

11. The inspector, on force account work, shall check over with the authorized representative of the contractor, at the close of each day's work, all items included therein.

Excavation and Foundation.

12. The inspector shall examine the finished excavation and determine whether it is down to the level specified and see that the material on which the foundation is to rest is satisfactory. If excavation is to go to rock he shall see that the surface of the rock is sound and completely exposed. He shall secure the approval of the engineer before permitting concrete to be placed.

13. The inspector shall see that the required number of piles are driven, each in its proper location and to the penetration required by the specifications.

14. The inspector shall check the piles in each group after driving and should report any pile which is materially out of line. He shall keep a record of:

(a) Method of driving;
(b) Type of pile;
(c) Length of pile;
(d) Diameter of pile;
(e) Total depth of penetration;
(f) Type and weight of hammer;
(g) Drop of hammer under last five blows;
(h) Penetration of pile under last five blows;
(i) Number of piles driven;
(j) Number of piles specified;
(k) Length of pile under cutoff.

15. The inspector shall see that the requirements of the specifications for placing concrete under water are strictly followed. In the absence of specifications, standard methods shall be used.

> Note.—In placing concrete under water it is highly important that extra precautions be taken by the contractor to execute the work properly in order that the concrete may have adequate density, be free from segregations, or the washing out of any of the materials, and to avoid exposure of freshly deposited concrete to running water.

Materials.

16. The inspector shall:

(a) Inspect and identify each lot of material upon its receipt on the work;
(b) Keep a careful watch of aggregates, especially if the source of supply is changed from time to time;

(c) Make frequent sieve analysis of aggregates;
(d) See that aggregates are clean and properly graded;
(e) See that aggregates are free from vegetable or animal matter as determined by the Colorimetric Test;
(f) See that materials are properly stored on the work;
(g) See that the cement is properly piled and tagged; and housed in such a manner as to protect it from the weather.

Aggregates should be piled in such a manner as to insure them against intermingling of foreign materials.

Metal re-inforcement should be stored preferably under shelter.

17. The inspector shall observe the following in selecting samples where tests of concrete materials are required under the specifications:

(a) Secure at least a 20-lb. sample of fine aggregate;
(b) Secure at least a 50-lb. sample of coarse aggregate;
(c) Give complete information concerning the sample (both on the inside and on the outside of the package);
(d) Samples shall be shipped in a strong sack or tight box; if a sack is used it shall be clean.

NOTE.—Tests on materials are worthless unless sampling is properly done. Particular care must be used in sampling aggregates under the following conditions, so as to get an average sample:

(a) Aggregates in undeveloped pits;
(b) Coarse aggregate in cars or stock piles.

METAL RE-INFORCEMENT.

18. The inspector shall see that the metal re-inforcement, before being positioned, is cleaned of mill and rust scale, and of coatings of any character that may destroy or reduce the bond. He shall reject re-inforcement appreciably reduced in section. If there is delay in deposting concrete he shall re-inspect the re-inforcement, and, when necessary, shall see that it is cleaned.

19. The inspector shall see that the required bending of bars is uniformly and accurately done.

NOTE.—Bends can be more uniformly and accurately made by machine than by hand. In case a considerable number of bars are to be bent alike, it is advisable to bend one, put it in position and check the bends before bending the others, to insure proper location and degree of bends.

20. The inspector shall see that the re-inforcement is of the specified size, bent in accordance with the specifications, and properly positioned in the forms before placing the concrete.

NOTE.—The inspector should not wait until the metal re-inforcement has been wired substantially in place before passing on the correctness of its position, but should inspect the re-inforcement during its placing.

The inspector should inform himself as to the details for the placing of the re-inforcement, especially as to size, length, bending, laps, and splices, and in case of doubt should secure instructions.

21. The inspector shall see that the bars are rigidly secured and that they are not displaced during the placing of the concrete.

22. The inspector shall see that all joints and splices in metal re-inforcement are located and made in full accordance with the specifications.

MIXING AND PLACING OF CONCRETE.

23. The inspector, in order to secure the proportions and mixing required by the specifications, shall:

(a) Check the capacity of wheelbarrows and hoppers, or other measuring devices, and see that the method of loading them will secure uniformly the specified proportions;
(b) Record total quantities of cement and aggregates used;
(c) See that the specified consistency is maintained throughout the work;
(d) See that the time of mixing is that specified;
(e) See that changes in proportions and consistency are carried out;

24. The inspector shall see that the concrete is placed in accordance with the specifications, and specifically that:

(a) The method of conveying the concrete is as specified; or in the absence of specifications that it is such as to avoid segregation or the formation of excessive laitance;
(b) The concrete is placed in the form as near as possible to the point of its final location and is not permitted to flow long distances in the form;
(c) It is properly spaded or tamped so as to fill the forms and surround the re-inforcement;
(d) It is properly spaded or forked to keep the coarse aggregate away from the forms so as to insure a smooth surface;
(e) The concrete is placed continuously without undue delay in each unit of the structure;
(f) Construction joints are made in accordance with the specifications; in the absence of specifications, that written instructions are obtained from his immediate superior.
(g) The concrete is poured in a continuous operation between construction joints;

(h) Where night work is necessary, adequate precautions as to lighting, etc., are taken;

(i) That at the end of each day's pouring in all work above the foundations, finishing strips are nailed in the forms in a horizontal plane and the concrete finished to them;

(j) That all laitance is removed and the surface thoroughly cleaned before depositing the new concrete;

(k) All anchor bolts, inserts, pipe sleeves, wiring, drainage pipes, flashings, conduits and other fixtures are in position as required by the plans and specifications before the concrete is placed; and that conduits and other equipment are built into the slab in such a manner as not to affect the position of metal re-inforcement or weaken the member at critical sections;

(l) The specified precautions are strictly observed when it is necessary to place concrete in cold weather;

(m) A prompt report is made where conditions require additional fittings or where installations not called for in the specifications should be in position before the concrete is placed.

25. The inspector shall see that the freshly-placed concrete is protected and kept moist in accordance with the specifications.

FORMS.

26. The inspector shall examine the forms before placing concrete and:

(a) Check same as to location and dimensions;

(b) See that the surface of the form is oiled or wetted in accordance with the specifications:

(c) Report immediately any seeming lack of strength in forms which would permit bulging or sagging between supports.

NOTE.—This pressure amounts to over 100 lb. a square foot of surface for each foot of depth, where the form is filled in 40 minutes or less. It is very difficult and usually impossible with the equipment available to force a form back into position after it has shifted or sagged out of place while being filled.

(d) See that the column and girder forms are straight and properly aligned;

(e) See that foreign materials, such as chips, blocks, and shavings are removed from forms before concrete is placed.

REMOVAL OF FORMS.

27. The inspector shall see that the forms are removed:
   (a) At the time and in the manner required;
   (b) With care so as not to injure the structure or mar the surface of the concrete.

28. The inspector shall see that:
   (a) The forms are not removed until the concrete has hardened sufficiently for safety;
   (b) Special precautions are taken to protect the concrete during cold weather.

NOTE.—Concrete hardens very slowly at a temperature below 50 deg. F. and setting action is scarcely perceptible at temperatures below 40 deg. F. The time during which the concrete has been exposed to temperatures below 40 deg. F. should be added to the periods ordinarily required in determining the time of removal of forms.

   (c) Any uncertainty as to the conditions affecting the removal of forms is promptly reported to his immediate superior;
   (d) Defects in the surface of the concrete are repaired promptly upon the removal of forms;
   (e) Heavy loading on green concrete is not permitted.

WEIGHING CONCRETE AGGREGATES ON HIGHWAY PAVEMENTS.

By R. W. Crum.*

As a result of experience in weighing concrete aggregates upon two concrete paving contracts in 1923, the Iowa Highway Commission has adopted the actual weighing of aggregates on road pavement construction as standard practice. No startling revolution in results on these two jobs was noted, but concrete as good or better than other concrete made of the same materials was produced, and some improvement in uniformity was noted. After the next construction season, we will have much more extensive data from the construction of our 1924 program, and from cores to be cut from the work done this year. Our experience has been entirely satisfactory thus far, and in view of the facts that: (1) the method entails no added cost, (2) no loss in time or decreased production occurs, (3) weighing is admittedly a much more accurate way of measuring granular materials than by loose volumetric measurements, and (4) that a valuable record of the amounts of materials used is secured, our only doubt is "why did we not take up this practice sooner."

Several conditions contributed to our desire to try out this method of handling aggregates.

Fig. 1 showing the daily variation in sieve analysis of the aggregates on a paving job, is typical of the results secured by the methods in customary use. The range in strength of the concrete is also shown. These data are the result of taking a sample from the road each day. Two 6 x 12-in. cylinders were made and sieve analysis made on a part of the sample after removing the cement by washing. The specified dividing line between fine and coarse aggregate was the sieve of 0.185-in. opening and the mix was based on 33 per cent of sand in the total aggregate. If the aggregates had been screened within the tolerance limit of 5 per cent and had all been accurately measured all of these lines would have fallen in a range of 5 per cent at the base point instead of the 20 per cent that was actually the case, with a corresponding improvement in the uniformity of the concrete. In attempting to better this condition, we found ourselves continually in controversy with contractors over the types of measuring boxes they wished to use, and many times we had to compromise on equipment which fell a long way short of our desires. We realized of course that we might improve this condition by making very rigid requirements in the specifications as to the measuring boxes, but it also occurred to us that it would also be possible to better control the mixtures by actually weighing the materials. This came the more naturally because the Iowa Highway Commission has for several years specified the proportions for paving mixtures by weight, and a part of our trouble has come from the

* Iowa State Highway Commission, Ames, Ia.

necessity of converting our weight proportions into volumes, and the further necessity of changing mixtures due to changes in unit weight of raw materials.*

We do not claim that in theory, weight measurements are any better than volumetric measurements, for if all the characteristics are known, the proportions can be specified in either way to yield the same result, but we do believe that measurements by weight will be better made and more consistently accurate.

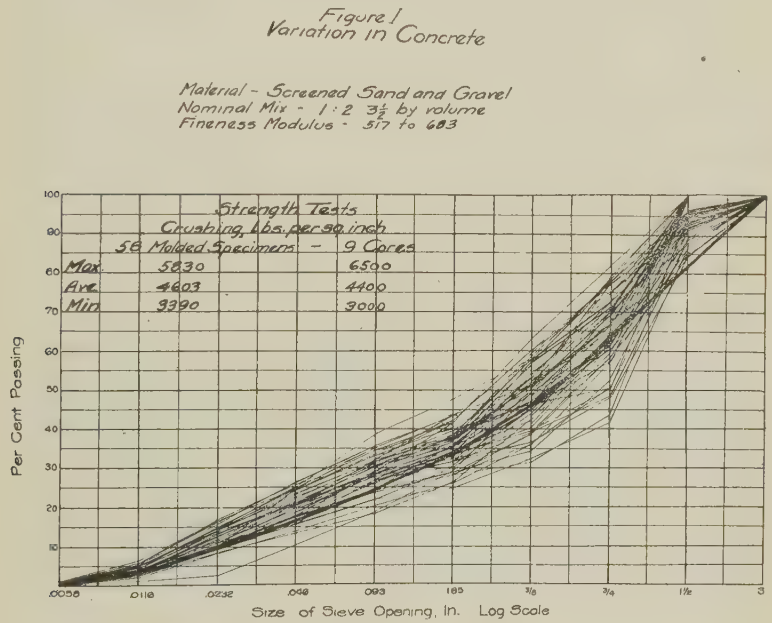


FIG. 1.—VARIATION IN CONCRETE.

Material, screened sand and gravel; nominal mix 1 : 2 : 3½ by volume; fineness modulus 517 to 683.

Another important factor in the uniformity is the moisture content of the aggregates, especially of the sand. There is no doubt that variations in moisture content have a large bulking effect upon the sand and therefore a corresponding effect upon volumetric measures. Of course, if the proportions are based upon dry materials, changes in moisture will be on the safe side and the concrete will be richer, and the contractor will be out some extra cement, but the concrete will not be uniform. I assume, however, that what we want is a uniform mixture that will not contain more cement than the contractor is led to expect. By using weights, corrections for moisture can be easily and frequently made.

*See A. S. T. M. Proceedings, Vol. XIX, p. 458, for a discussion of Iowa conditions.

Consider the possible effect of moisture upon a typical case. A 1: 2: 3½ mixture of dry materials would require, to make concrete 80 per cent dense, 1.42 bbl. of cement per cubic yard. If the sand used should contain 2 per cent of moisture, the result would be the use of 1.506 bbl. of cement per cubic yard of concrete. On the other hand, had the materials been weighed in the weight proportion equal to the 1: 2: 3½ of dry mate-

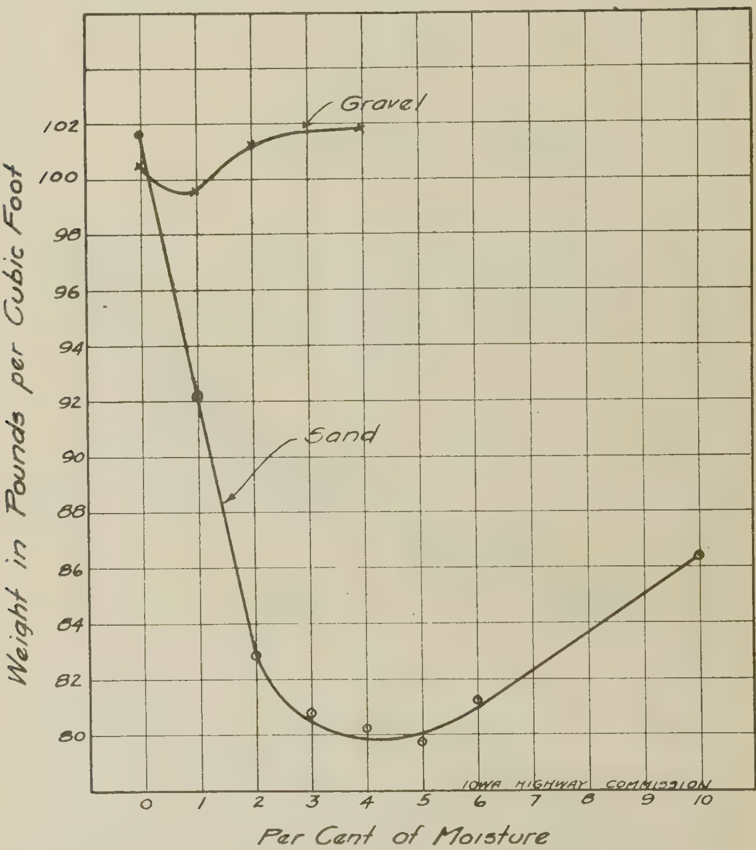


FIG. 2.—MOISTURE WEIGHT CURVE, SAND AND GRAVEL MEASURED LOOSE.

rials, the cement content for the 2 per cent moisture would be 1.43 bbl. per cubic yard, and even this increase could be lowered by correcting the weights for the moisture content. These figures are based on the following unit weights, from actual tests, as shown in Fig. 2.

Sand dry 101.6 lb. per cu. ft.
Sand 2 per cent moisture 83 lb. per cu. ft.
Gravel, dry 100 lb. per cu. ft.

A valuable result from weighing the materials is an accurate record of the amount of material actually used in the structure. Since such materials are customarily bought and sold by weight, such a record should be of great value, especially to the contractor. At several times in the past few years, settlements between the Highway Commission, contractors, material producers, and carriers would have been greatly facilitated had such a record been available. In one case, a paving contractor was billed for sand and gravel and freight at such a rate that every cubic yard of concrete he made would have contained 3900 lb. of sand and gravel. In the proportions specified, this would have meant the production of concrete

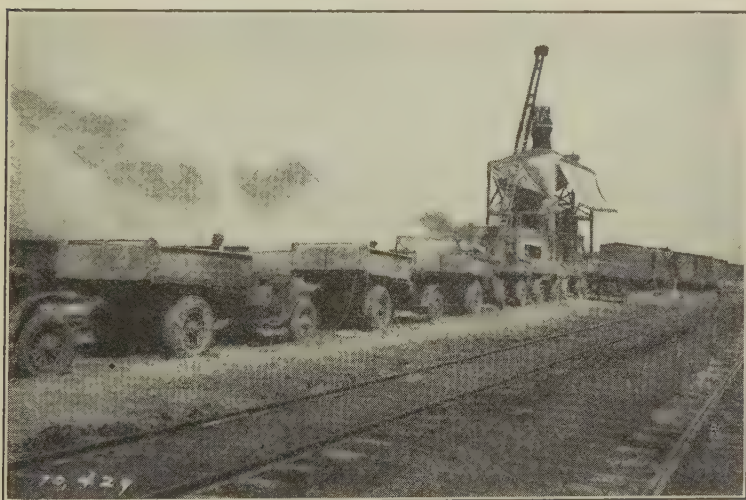


FIG. 3.—CENTRAL MIXING PLANT OF THE E. F. LYTLE CONSTRUCTION CO., WOODBURY CO., IOWA.

105 per cent dense. The producer of the material recognized this absurdity, and settled for a more reasonable amount, but this settlement was on a guess and not based on facts as it would have been had the material been weighed when mixed.

As in the case of moisture variation other conditions, such as changes in grading, may make it desirable to make slight changes in the mixtures. At the present time such changes are seldom made, but when materials are weighed it is very easy to make such changes and thus take advantage of another opportunity to produce more uniform concrete.

We have felt that the advantages mentioned above are obvious and in the light of our experience upon actual construction, we feel that by using this system a much more favorable condition will be set up for securing

uniform results than has heretofore been the case. All paving contracts on the Iowa Primary road system for 1924 will be let on this basis. About 100 miles is already under contract.

Experience.—On two contracts completed last season the aggregates were weighed. One contract involved construction of about 20 miles of concrete road slab in Woodbury County, Iowa, 18 and 20 ft. wide and 8 in. thick, the other about 2 miles in Buchanan County, Iowa, 18 ft. wide, 8 in. thick. In both cases the aggregates were sand and screened gravel. The characteristics of the materials were as follows:

SAND.

	Woodbury Co.	Buchanan Co.
Per cent passing Sieve No. 4	97.6	97.8
" " " " " 8	79.5	82.0
" " " " " 14	56.7	65.3
" " " " " 28	38.1	39.7
" " " " " 48	11.5	9.8
" " " " " 100	1.1	1.0
Per cent of silt	1.0	0.7
Percentage of rotten stone	...	0.5
Percentage of shale	...	0.1
Weight per cubic foot	...	94 lb.

GRAVEL.

	Woodbury Co.	Buchanan Co.
Per cent passing Sieve 1½ in.	80	84
" " " " ¾ "	55.5	43
" " " " ⅜ "	26.7	14
" " " " No. 4	5.5	2
Weight per cubic foot	...	108 lb.
Percentage of rotten stone	1.7	1.0
Percentage of shale	0.1	0.0

The Woodbury County installation consisted of an ordinary bottom dumping hopper suspended by means of a lever weighing system from the overhead bins, the load being read by means of a Toledo dial scale. The mixture by weight was 1 : 2.17 : 2.17, and each batch contained fourteen bags of cement, 2850 lb. of sand and 2850 lb. of gravel. The sand and gravel were weighed separately. The cement was not weighed, but taken as 94 lb. per bag. This was a central mixing plant, the weighing hopper discharging into a 2 yd. mixer. Figs. 3 to 7 are pictures of this installation.

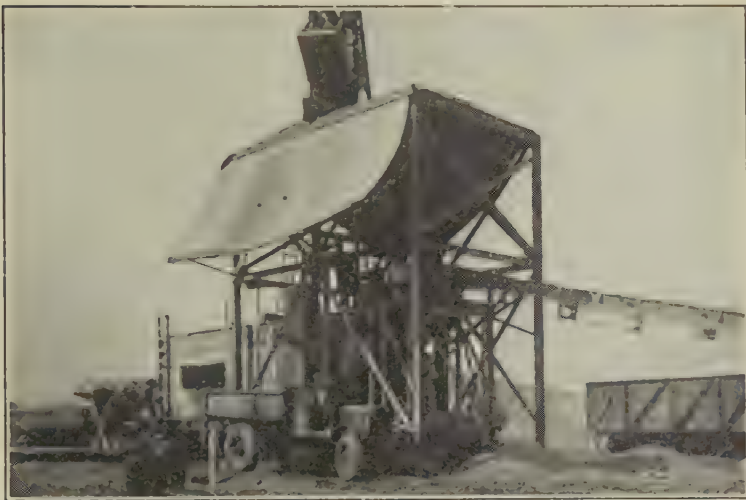


FIG. 4.—SAME PLANT AS SHOWN IN FIG. 3.

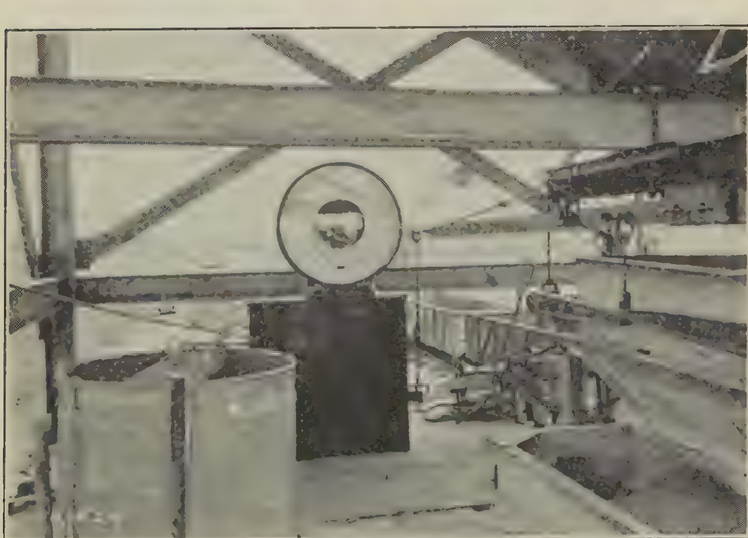


FIG. 5.—WASHING HOPPER AND TOLEDO DIAL SCALE.

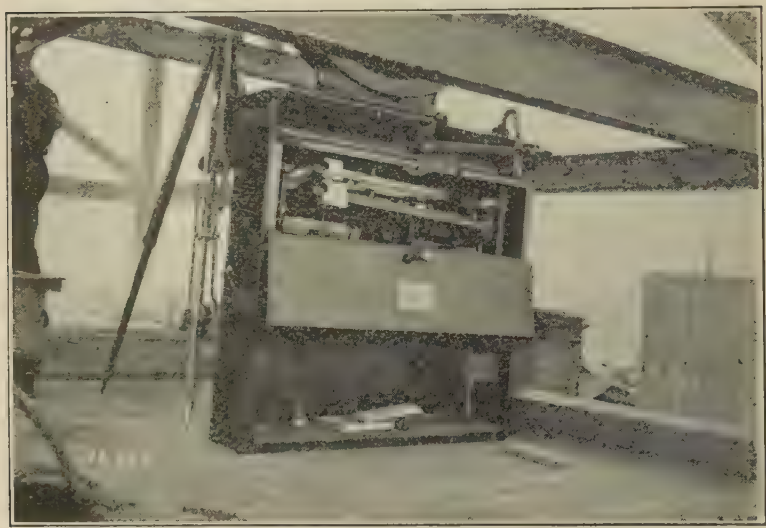


FIG. 6.—TARE BEAMS OF TOLEDO DIAL SCALE.

FIG. 7.—PAVING OPERATION, E. F. LYTLE CONSTRUCTION CO., WOODBURY CO., IOWA. MAXIMUM HAUL OF CONCRETE 10 MILES.

In Buchanan County, the contractor used an industrial railway outfit with a Koehring 34E mixer on the grade. The batch boxes were loaded from overhead bins having eight measuring hoppers, four for sand and four for gravel. Each hopper rested directly upon the weighing platform

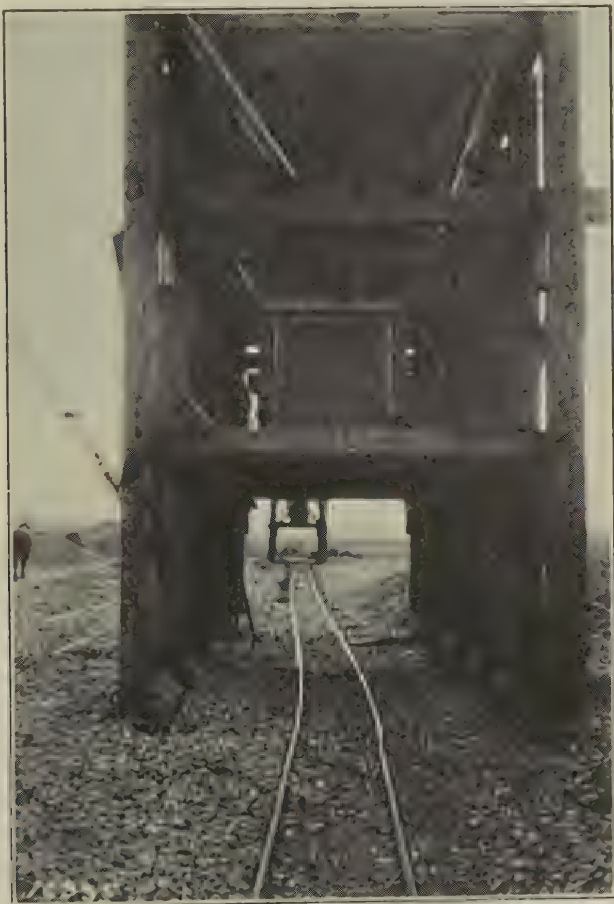


FIG. 8.—INDUSTRIAL RAILWAY PLANT, NORTHERN STATES CONTRACTING CO., BUCHANAN CO., IOWA.

of a Fairbanks scale of 8000 lb. capacity. The size of each batch was, 10 bags cement, 1607 lb. sand and 3262 lb. gravel. This installation was very satisfactory and caused no extra labor cost or loss of time. So far as we can tell, the use of the weighing equipment has added practically nothing to the cost of the pavement. Figs. 8, 9 and 10 are pictures of this installation.

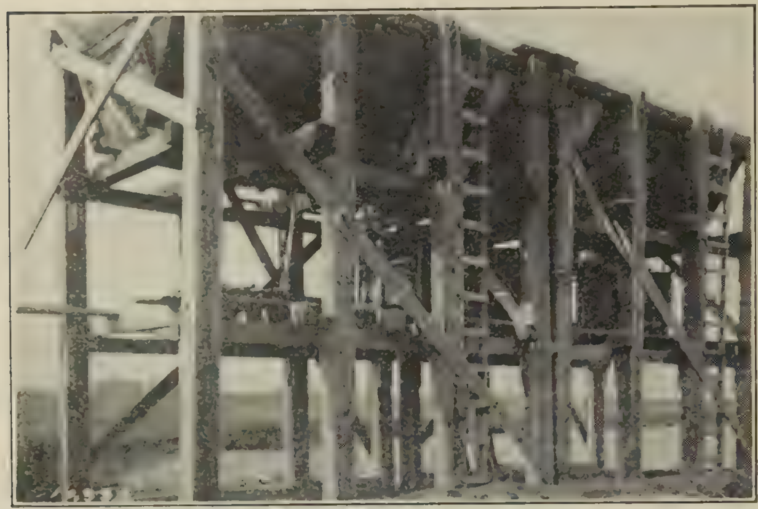


FIG. 9.—SAME PLANT AS SHOWN IN FIG. 7.

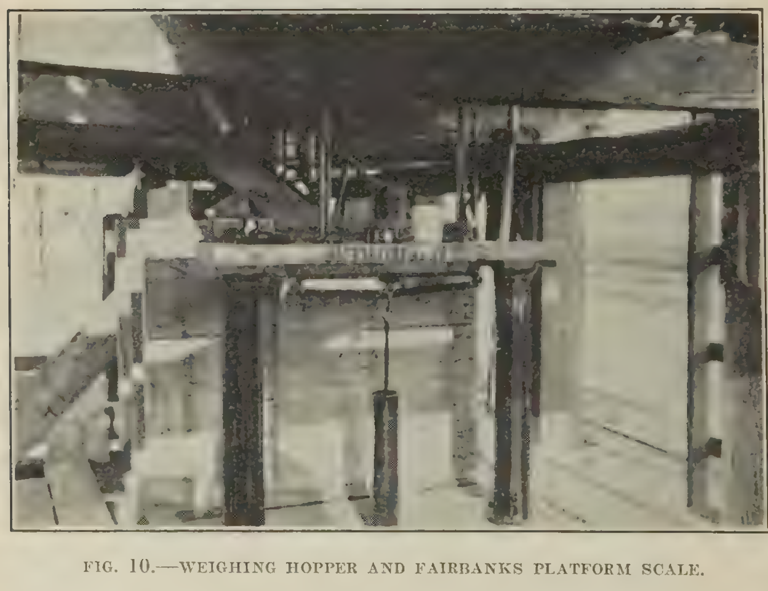


FIG. 10.—WEIGHING HOPPER AND FAIRBANKS PLATFORM SCALE.

Results.—Since there are several other factors affecting the uniformity of concrete, it is not to be expected that this change in method of measuring materials will entirely revolutionize the quality, but since this is a very important factor some improvement in uniformity is to be expected. Reliable comparative data as to the strength of the concrete will not be available until test cores are taken from these pavements next summer. However, 6 x 12-in. cylinders were made daily and the strength tests of these cylinders are of interest. These data are shown on Figs. 11 and 12. Data are shown for the work upon which the aggregates were weighed and for corresponding volumetric measured jobs constructed of the same materials and under similar conditions.

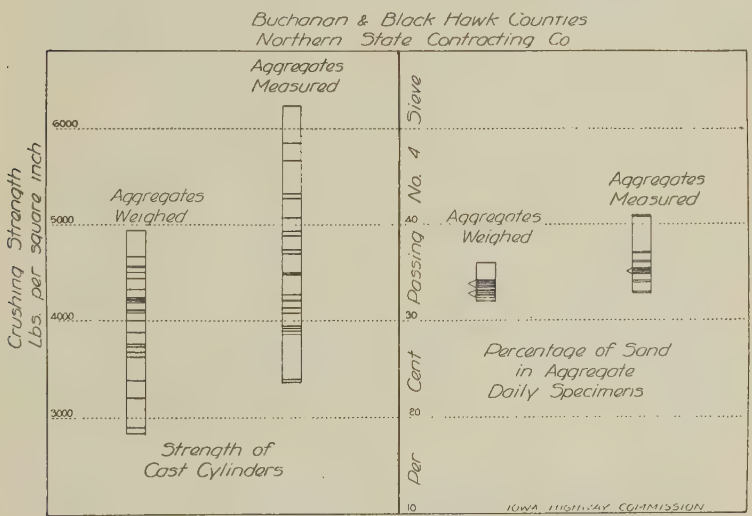


FIG. 11.—DAILY VARIATION IN CONCRETE.

In the Buchanan County work, a considerable less variation is noted on the work where the aggregates were weighed. In Woodbury County, no improvement in uniformity is shown by the cylinders. An explanation of this will be discussed later.

Another method of estimating the uniformity is by means of sieve analysis after the concrete is made. Every day a sample of fresh concrete was taken from the road, the cement removed by washing, and the aggregate separated by a No. 4 screen into its component parts of fine and coarse aggregate. If the aggregates had been perfectly screened to begin with, and the measurements had been exactly right each sample would have yielded the relative amounts of fine and coarse aggregate upon which the mixture was based. Figs. 11 and 13 show the actual variation. In the Woodbury County case, only a slight improvement may be noted.

FIG. 12.—DATA FROM PAVEMENT CONSTRUCTION, WOODBURY CO., IOWA. STRENGTH OF CAST CYLINDERS.

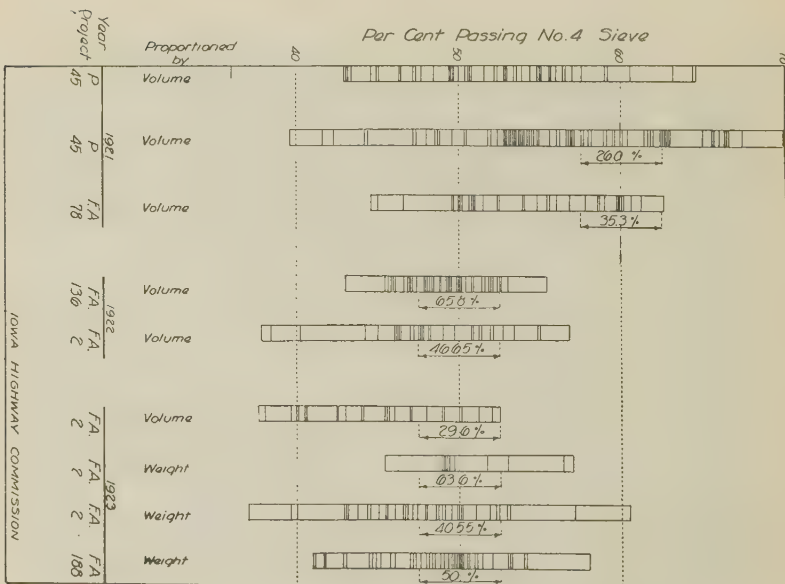


FIG. 13.—DAILY VARIATION PERCENTAGE OF SAND IN AGGREGATE, WOODBURY CO.

Since the screening tolerance of the specifications is 5 per cent, the significant figure is the percentage of the samples which fall within the tolerance limit which ranges from 2½ per cent below to 2½ per cent above the base ratio of fine to coarse aggregate.

In the case of the Buchanan County work, a decided improvement can be noted although the data is more meager.

The indication from these data is that, if the materials are well screened, satisfactory increase in uniformity can be secured by weighing the aggregates, but that if materials are poorly screened the advantage from weighing the aggregates will probably be lost.

The Buchanan County materials were thoroughly screened. The Woodbury County materials were from an overloaded plant and were poorly screened.

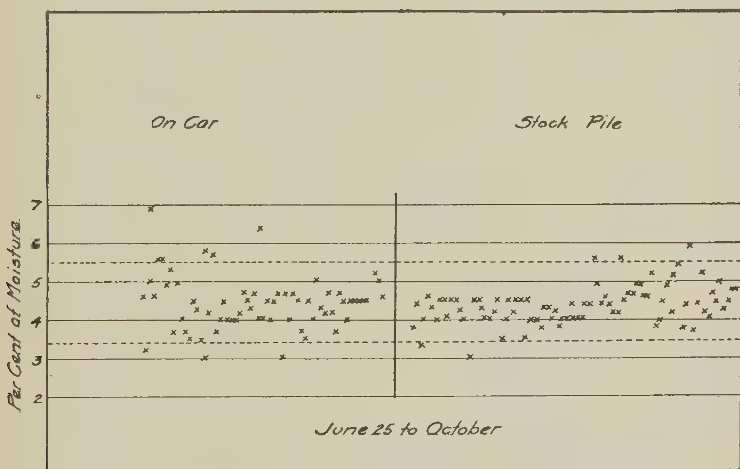


FIG. 14.—MOISTURE IN SAND, PAVEMENT CONSTRUCTION, WOODBURY COUNTY, IOWA.

PLANT INSPECTION RECORD.

	No. of cars of gravel represented	No. of Tests	Per cent passing No. 4 screen Maximum	Average	Minimum
Buchanan Co.	120	101	3.00	2.43	1.00
Woodbury Co.	1057	342	18.00	7.95	2.60

Moisture.—In Woodbury County, determinations of the amount of moisture in the sand was made about four times each day, and in the gravel about once each day, and the weights were corrected for moisture content. These data, shown in Figs. 13 and 14, are interesting and show the range to be expected. In this case there appears to be very little difference in the range of moisture content between the stock pile, and the

individual cars as they arrived. The sand was loaded very wet and the railroad haul was about 40 miles. It is interesting to note in the case of the sand that the moisture content ranged in general from about 3½ to 5½ per cent. The effect of this variation on the Woodbury County mixture would cause the cement content of the concrete to vary only from 99.5 per cent to 100.5 per cent of the estimated amount of cement and it would therefore appear that under given conditions a moisture correction could be established from which only very occasional departures would need to be taken. The same thing is also true of the gravel, the moisture range being from 1.5 per cent to 3.5 per cent.

The weighing of the aggregates in itself is undoubtedly an improvement in the detail of measuring, but in order to secure the full benefit

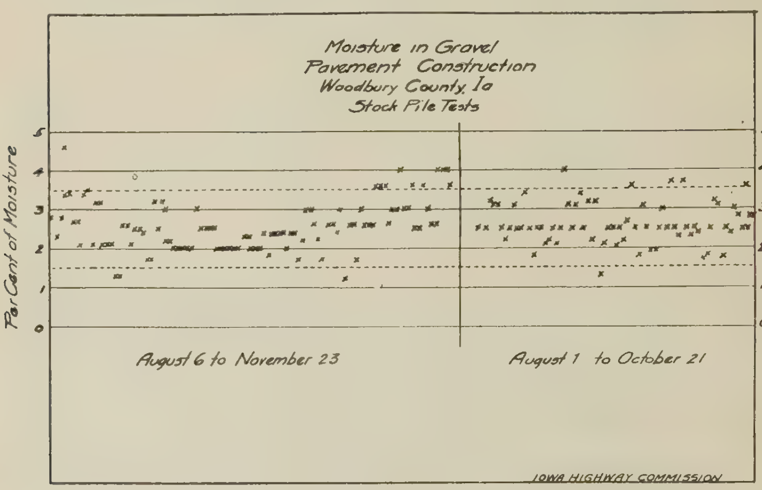


FIG. 15.—MOISTURE IN GRAVEL. PAVEMENT CONSTRUCTION, WOODBURY COUNTY, IOWA. STOCK PILE TEST.

possible, it will be necessary to increase our care in the control of other details, especially the grading of the coarse aggregate, the separation between fine and coarse aggregate, and the amount of water used in mixing. It is of course of no avail to specify such accurate methods of measuring if the concrete is allowed to vary from dry to sloppy consistency. Although we have not yet tried, it seems to the writer, that if the moisture content of the aggregates as used is known, it should be entirely possible to specify a definite water-cement ratio and secure concrete of uniform consistency. We intend to try and work this out in a practical way this summer.

As respects the separation of the fine and coarse aggregates, it was noted above, in the case of the Woodbury County work, that the wide variation in the percentage of sand found in the coarse aggregate, obviated to some extent the benefit of weighing. It will probably not be practicable

to use a screen tolerance of less than 5 per cent, but we believe that a 5 per cent tolerance can and should be rigidly enforced. As a matter of fact, 5 per cent has been our specified limit for the last four years, and we have generally had successful results. We were rather unusually unfortunate in the case noted.

Another characteristic of coarse aggregates, especially when handled to and from large stock piles is segregation in various parts of the pile of materials varying widely in grading. This will of course under certain conditions cause a decided lack of uniformity no matter how the materials are measured. The particular condition under which this effect is the worst, is when the mixture is such that the mortar is just barely sufficient to fill the voids in a well graded coarse aggregate. The common 1: 2: 4 is such a proportion. In most cases, a decided improvement could be made by stationing a competent inspector at the stock pile to make sure that the materials are handled in such a way as to secure a minimum of segregation, but there are at least two methods of curing the condition more effectively. One would be to have the materials furnished in several sizes and have them carefully mixed by weight as is now done in the case of asphaltic concrete mixtures. The other, and this is a practice that has been used by the Iowa Highway Commission, is to so design the mixture that there will be an excess of mortar over the voids in the coarse aggregate, in which case most authorities are agreed a considerable variation in the coarse aggregate would have little effect upon the quality of the concrete.

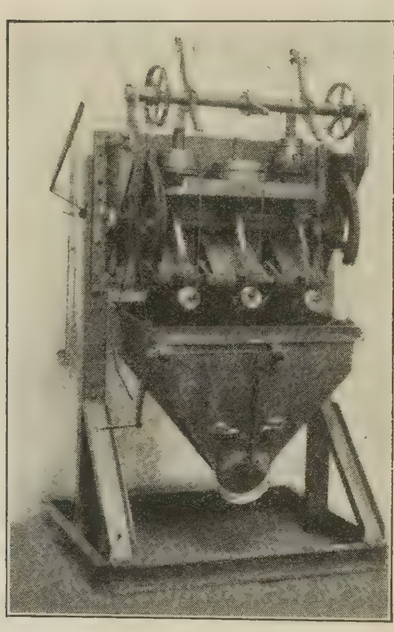
I believe it is generally agreed that concrete as made in most cases at the present time is sadly lacking in uniformity, and that improvements in methods of control are a vital necessity. This paper presents such observations as we have made on the results of what appears to us to be one obvious method of improving control. We hope that this method will be tried at other places and on other classes of work. In conclusion, I wish to emphasize again the importance of the problem of control of concrete fabrication, and the necessity of making careful investigation of all possibilities that may appear.

DISCUSSION.

(By Letter.)

N. C. Johnson.—I want to commend especially the first and second sentences of the second paragraph of the paper. In these two sentences the author has touched the very heart of the somewhat lamentable concrete art as it is practiced, virtually under the control of a labor force.

I note from the illustration that the author used standard scales; whereas, as will be seen by the view herewith of an improved and more accurate weighing device in ⅛-in. scale model, I use a self-contained



apparatus carrying gates and all. In the improved form it may be used on any angle, and is so sectionalized as to be knocked down for storage.

I am not an expert in the weighing art, but I am given to understand by a scales manufacturer that the large lever arm scales, such as the author seems to be using, show a very material inaccuracy. I am advised that the only really accurate scale is one having equal lever arms; and to approach this accuracy, the new apparatus carries a one-to-three lever arm for sand and stone, and an equal lever arm for cement where this is to be weighed.

I agree with the author that it is better to use bag cement than to weigh it.

With a greater degree of accuracy, I wonder whether it is necessary to calculate the moisture content? My own data seems to show that the actual silica error is about 53 lb. per yard with greater or less commercial quantities of water, as found in commercial materials. This error is practically negligible; and with such weighing it is found that a fixed quantity of water will give a fixed consistency.

ECONOMIC VALUE OF ADMIXTURES

By J. C. Pearson * and Frank A. Hitchcock.*

By admixtures we mean substances other than cement, aggregates and water which are added to concrete mixtures for the purpose of imparting to the latter certain improved qualities. Admixtures of many different kinds have been marketed and used, but we are more accustomed to think of the term as applying to finely divided materials, of which hydrated lime is probably the most familiar example. Such admixtures are claimed to improve the plasticity or workability of concrete and mortars, and there is no doubt that this effect is pronounced in the case of mortars. Nevertheless it is a fact that this property is not readily susceptible of measurement, and so far as we are aware, no method has been devised for determining which of two mortars, nearly alike in plasticity or workability, is actually superior in this respect. (To avoid confusion we shall consider plasticity and workability to have the same meaning throughout this discussion.)

This question is of considerable economic importance, for if by any means a mortar or concrete mixture can be made more plastic and still serve its ultimate purpose satisfactorily, it means a saving in labor cost, and a more uniform product under given labor conditions. We believe that the importance of this question has been grossly under-estimated, probably for the reason that there has been no answer to it.

About two years ago the authors seriously took up the problem of finding a method of comparing the workability of concrete mixtures. Previous to that time abortive and unsuccessful attempts to measure this quality had led to a single conclusion—that the difference in workability was there, and that neither the slump test nor the flow table furnished any information on this elusive property. We knew, as everyone knows, that a 1: 2: 4 mixture of a given slump or flow or "consistency" was easier to mix and place in forms than a 1: 3: 6 mixture of similar consistency, but what was it that one could put his finger on and *measure*, to demonstrate that the former was easier to handle and place in forms? We tried many methods—direct methods, such as measuring the amount of work necessary to move a concrete mixture from one place to another; but this failed because we could not tell when the transfer had been completed. We tried

* U. S. Bureau of Standards, Washington, D. C.

indirect methods, such as measuring the tendency of the concrete to retain its mixing water in a filter; we had fair success with methods of measuring segregation, but these failed when the concrete approached anywhere near the same degree of workability. After much study we came to the conclusion that the most obvious difference between concretes of different workability was the tendency of the harsher mixes to allow their solid constituents to settle and pack at the bottom of the mass. Then followed a search for a method of measuring this packing, and finally we devised a simple penetration test which was described last year before the American Society for Testing Materials.[1]

PENETRATION TEST FOR WORKABILITY.

This test is only a partial solution of the problem, but it is of interest because it has enabled us to demonstrate and compare the effects of admixtures on workability.

The apparatus is shown in Fig. 1, in which *M* is a 6 in. by 12 in. cylindrical mold with a tight bottom mounted upon a small table which can be raised and dropped about one-tenth of an inch by means of a cam. A spider *F* is mounted on top of the mold carrying a sleeve *S* into which a close fitting steel rod about 20 inches long can be introduced in alignment with the axis of the mold. In our most recent experiments the rod is tapered to a 45° cone on its lower end, and is graduated in such manner that the depth of the lower end below the top of the mold is read directly.

The batch of concrete to be tested is placed uniformly in the mold and first compacted by 30 drops of the table, then the rod is inserted in the sleeve and lowered gently into the concrete until it comes to rest under its own weight. The cam shaft is then turned and the mold full of concrete is successively raised and dropped in such manner as to allow a reading on the rod after each impact. The test is regarded as complete when the rod has penetrated the concrete to a depth of 10 inches. From the data of the test a penetration curve is obtained of the type shown in Fig. 2. It is obvious that the richer or more plastic the concrete mixture, the more readily will the rod penetrate to the prescribed depth, and the number of impacts required to cause the rod to reach the 10-in. mark is therefore a rough index of the workability of the mixture.

In actually making comparative tests it is found desirable to take as an index of workability, not simply the number of impacts required to produce a penetration of 10 inches, but the area under the penetration curve.

---

[1] A Penetration Test for the Workability of Concrete Mixtures with Particular Reference to the Effects of Certain Powdered Admixtures, by J. C. Pearson and Frank A. Hitchcock, Page 276, Vol. 23, Part II, A. S. T. M. Proceedings, 1923.

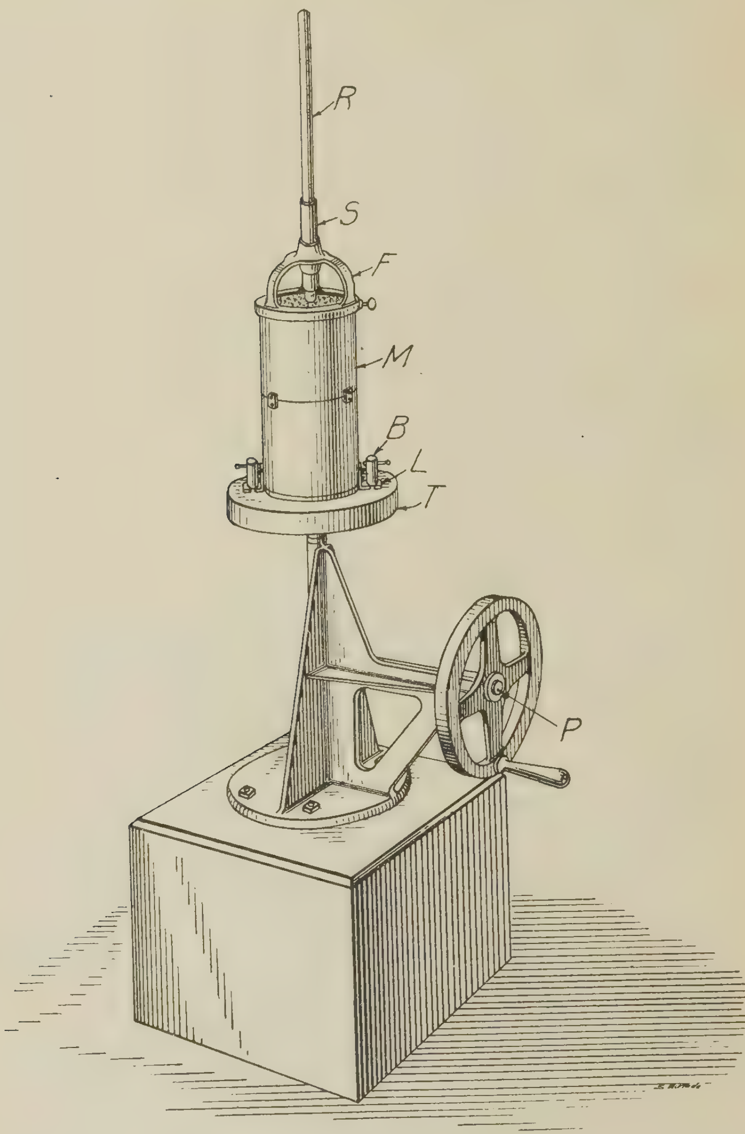


FIG. 1.—PENETRATION APPARATUS.

This procedure is quite arbitrary and is adopted in order to obtain a figure from the entire curve, rather than a single point on the curve. The workability figures so derived are high for the harsh working mixtures, and low for the plastic mixtures. It is important to note that these workability figures have no real significance in themselves, and are not directly proportional to the *amount* of *work* required in mixing, handling and placing the concretes from which they are derived. They merely indicate the order

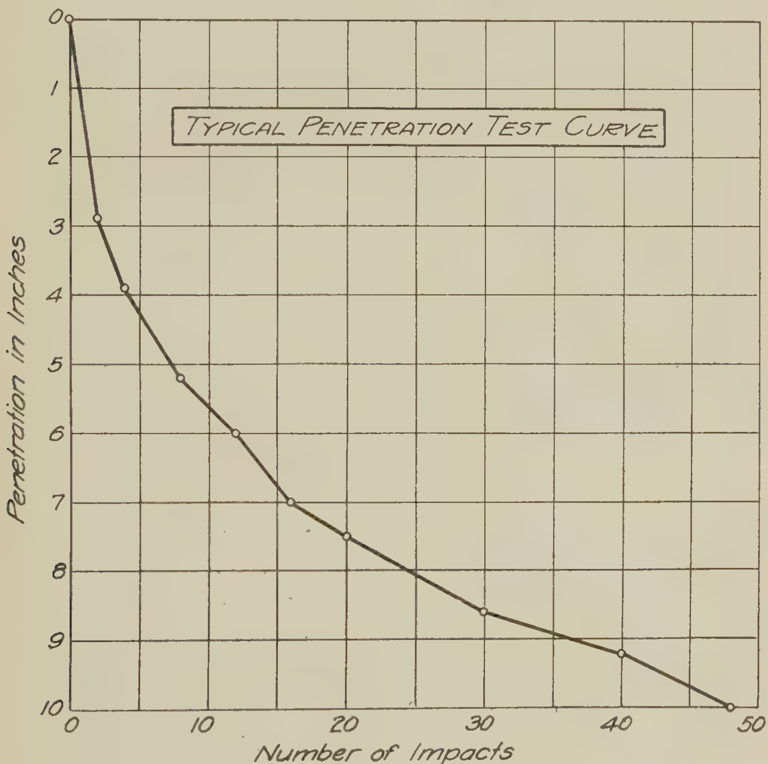


FIG. 2.

of workability of different mixtures, and permit us to determine whether one mixture is about the same or better or worse than another. In the long run these figures have been found to be reliable indices of the workability of concrete mixtures in so far as we are able to judge this quality. and we have therefore accepted the test as a partial solution of the problem—useful until some better test is devised. We have added to this paper a brief supplementary discussion of the defects of the penetration test, and of the possibilities of arriving at a more general solution of the problem.

Effect of Admixtures on Properties of the Finished Concrete.

Having established by means of the penetration test a marked difference in the workability of concretes gauged to the same consistency by the flow table, we believed it desirable to confirm the earlier tests by a somewhat more extensive investigation, and at the same time to examine rather closely some of the effects of admixtures on the finished concrete, in order that the evidence for and against the use of admixtures may be properly weighed. For example, the strength of the richer concretes is generally reduced slightly by the use of admixtures. It is important to know whether this is really serious, and whether and to what extent the reduction in strength is offset by the gain in other respects.

We therefore planned this investigation in such a way that the data obtained would help to answer the following questions:

1. Were our original determinations of the effectiveness of different admixtures correct—that is, could we check our conclusions by using those admixtures in different proportions in such manner as to give concretes of the same workability?

2. Is the reduction in strength of concrete containing admixtures in small proportions serious?

3. Is the effect of admixtures on strength of concrete more favorable under average job conditions than under standard laboratory conditions?

4. Is the settlement or shrinkage of the plastic concrete greater when admixtures are used than when they are not used?

5. Do admixtures increase the "yield" of concrete sufficiently to effect an appreciable saving in cost?

6. To what extent are the voids or the density of concrete affected by the use of admixtures?

7. Do admixtures tend to prevent loss of water from the concrete by evaporation under dry or partially dry storage conditions?

8. Is the use of admixtures considered advantageous and advisable, all things considered?

With these questions in mind the following program of tests was planned and carried out.

Admixtures—Hydrated lime, kaolin, and diatomaceous earth (celite), which had been used in the earlier investigation were used in this. From the previous tests we had estimated the effectiveness of these materials as approximately proportional to the numbers 1, 2, and 3, when used in equal parts by weight. In these tests, we therefore took of hydrated lime 5, 10 and 15 per cent, of kaolin 3 1/3, 6 2/3 and 10 per cent, of celite 1 2/3, 3 1/3 and 5 per cent, by weight of the cement. We also selected three ad-

mixtures of the calcium chloride type which were known to possess different characteristics. Two of these were proprietary compounds, and one was a 4 per cent solution of commercial chloride.

Basic Concrete Mixtures—For convenience we decided to use the weight proportions 1: 1½: 3, 1: 2: 4, 1: 2½: 5 and 1: 3: 6. All three proportions of admixtures were used in the two leaner mixtures, but the highest percentages of admixtures as given in the foregoing paragraph were omitted from the two richer mixtures.

Coarse Aggregates—Potomac River Gravel, washed, screened and recombined in the proportions of 50 per cent ¼ in. to ½ in. and 50 per cent ½ in. to ¾ in. was used in all mixtures without variation.

Fine Aggregates—Two lots of Potomac River sand were used. The first lot was prepared for the entire series by mixing and sacking in damp condition to insure uniformity. This sand had a fineness modulus of 2.91 and gave a very harsh working mixture in the 1: 3: 6 combination. In fact some of the workability readings on this mixture were lost through failure of the rod to penetrate to the 10-in. depth. On this account the sand was discarded after 60 per cent of the specimens had been made, and the remainder of the program was carried out with a new lot of prepared sand having a fineness modulus of 2.71. This second lot of sand gave a distinct improvement in workability, but did not appreciably diminish the strength of the concrete. The sieve analyses of the two sands were as follows:

Sand.	Retained on Sieve, per cent.						Fineness Modulus.
	4	8	16	30	50	100	
Coarse	2.5	19.0	34.6	53.2	84.8	97.2	2.91
Fine	3.5	17.3	29.4	45.1	80.1	95.8	2.71

Cement—A well-known brand from the Lehigh district, meeting the specifications, was carefully prepared and used in all batches.

Age at Test, Consistency and Storage—All test pieces were broken at 28 days. The flow was held as closely as possible throughout the series to a figure between 95 and 100, a good reinforced concrete consistency. One complete set of cylinders was stored in the damp closet, one complete set out of doors on the north side of the laboratory, and one complete set inside in the air of the laboratory.

Special Determinations—In addition to the usual determinations of quantities of materials entering into the concrete, from which the density and yield values are calculated, the rate of drying of the cylinders under

the three storage conditions was determined by careful periodic weighings; also the amount of settlement or plastic shrinkage was determined by striking off the freshly molded cylinders carefully, and measuring the settling or subsidence after initial set had occurred and just before capping.

Special Precautions—In view of the fact that the investigation was designed to bring out small differences in such factors as density, yield, strength, etc., special precautions were taken to insure uniformity and dependability of the results obtained. To this end each of the 750 batches were prepared by weighing the different kinds and quantities of material entering into each separately; predeterminations were made of the quantity of water required for the desired flow; all specimens were made in the forenoon and in order to allow the proper interval before applying the cement caps; and on removing the cylinders from the molds at a little more than 24 hours all caps were brought to plane surfaces with a straight, hard steel, cutting edge.

It will be seen that there were 46 different mixtures used in the series, and as double the number of cylinders were made from the four basic mixtures a complete round of the mixtures required the making of 50 batches. With the unusual number of measurements, embodying the preparation of the batches, the flow test, the penetration test, the weighing of the unit volume of the concrete and the making of the cylinders, we were able to make but 25 specimens per day. Thus, two days were required for a complete round of the mixtures, 15 batches of each kind being made up on 15 different days, 5 for each of the three storage conditions. As mentioned above, 30 batches were made from each of the four basic mixtures.

RESULTS OF THE TESTS.

It seems preferable to include the data of the tests in a single table, and to bring out the deductions from these data by reference to this table (Table I).

Fine Aggregate—As some doubt arose as to the validity of averaging the results from the two fine aggregates, data corresponding to those given in Table I were worked out separately for the coarse sand and the fine sand. Except in the case of the workability determinations, all values for the two sands were found to be in close agreement, and the data in Table I are therefore the weighted average of like batches, differing only in this one respect. To indicate how much liberty was taken in averaging results from the two sands, the averages of all the data from all the mixes for the two sands are given at the bottom of Table I. It will be noted that the differences in the data for the coarse and fine sands, except in the case of the workability figures, are hardly appreciable. We may call attention, however, to appreciable differences in the average loss of weight and the strength of the specimens stored in the laboratory. These differences are not attributed to the difference in the sands, so much as to the fact that the prevailing relative humidity was lower during the latter part of the work when the change to the finer sand was made.

Table I.—Results of the Tests

Mixture	Admixture	% Water	Flow	Density	Yield	% Cement/Concrete Absolute	% Cement/Concrete by Volume	c/(v+c)	Water-Cement Ratio	Plastic Shrinkage	Loss in Weight: Damp Closet	Loss in Weight: Outside	Loss in Weight: La… a…
1:3:6	0	9.6	97	.781	1.275	6.66	14.02	.233	1.45	0.96	−0.22	2.60	4
	5L	9.6	99	.785	1.281	6.65	13.98	.236	1.44	0.88	−0.27	2.47	4
	3⅓K	9.6	99	.782	1.282	6.65	13.98	.234	1.44	0.96	−0.28	2.43	4
	1⅔C	9.8	97	.779	1.285	6.63	13.94	.231	1.46	0.92	−0.22	2.66	4
	10L	9.7	99	.785	1.289	6.60	13.89	.235	1.46	0.92	−0.24	2.44	4
	6⅔K	9.8	98	.780	1.288	6.61	13.90	.231	1.46	1.04	−0.34	2.56	4
	3⅓C	10.1	96	.776	1.293	6.60	13.86	.228	1.51	1.00	−0.28	2.70	4
	15L	9.9	98	.784	1.298	6.56	13.80	.232	1.48	0.96	−0.32	2.58	4
	10K	9.9	97	.779	1.296	6.58	13.82	.229	1.48	1.12	−0.27	2.60	4
	5C	10.5	96	.770	1.306	6.52	13.72	.221	1.57	1.04	−0.28	2.69	4
	A	9.5	99	.787	1.280	6.65	13.98	.237	1.43	1.00	−0.22	2.13	3
	B	9.3	100	.785	1.272	6.70	14.08	.237	1.41	0.92	−0.36	2.13	3
	C	9.1	96	.777	1.286	6.63	13.92	.229	1.36	0.96	−0.35	2.08	3
1:2½:5	0	9.4	103	.783	1.297	7.88	16.56	.266	1.20	0.96	+0.02	2.16	3
	5L	9.5	100	.786	1.304	7.85	16.48	.269	1.21	0.92	0.00	2.12	3
	3⅓K	9.6	98	.782	1.305	7.84	16.47	.265	1.22	1.04	0.00	2.70	4
	1⅔C	9.8	99	.779	1.308	7.83	16.44	.262	1.25	0.96	+0.09	2.65	4
	10L	9.8	99	.784	1.316	7.77	16.34	.264	1.25	0.92	−0.07	2.46	3
	6⅔K	9.7	98	.780	1.314	7.79	16.36	.263	1.25	1.12	−0.03	2.45	4
	3⅓C	10.1	96	.774	1.320	7.74	16.28	.255	1.30	1.04	+0.06	2.87	4.
	15L	10.0	97	.782	1.328	7.71	16.20	.261	1.28	1.00	0.00	2.35	4.
	10K	10.0	98	.778	1.322	7.73	16.26	.258	1.28	1.08	−0.02	2.88	4.
	5C	10.8	97	.768	1.333	7.68	16.14	.248	1.37	1.08	−0.02	2.80	4.
	A	9.4	100	.787	1.304	7.85	16.49	.269	1.20	0.96	+0.05	2.04	3.
	B	9.1	99	.787	1.290	7.93	16.64	.271	1.17	0.96	0.00	1.85	3.
	C	8.9	100	.778	1.306	7.83	16.46	.261	1.14	1.00	+0.02	1.71	3.
1:2:4	0	9.6	98	.783	1.331	9.61	20.19	.307	0.98	1.08	+0.20	1.94	3.
	5L	9.6	94	784	1.342	9.54	20.02	.307	1.01	1.08	+0.14	2.14	3.
	3⅓K	9.6	100	.780	1.342	9.53	20.02	.302	1.01	1.12	+0.06	2.09	3.
	1⅔C	9.9	100	.776	1.347	9.48	19.94	.297	1.04	1.12	+0.02	2.28	3.
	10L	10.0	99	.779	1.362	9.39	19.75	.298	1.05	1.00	+0.09	1.97	3.
	6⅔K	9.9	98	.776	1.356	9.43	19.81	.296	1.04	1.04	+0.03	2.19	3.
	3⅓C	10.6	98	.772	1.366	9.35	19.67	.290	1.12	1.08	+0.05	2.30	3.
	A	9.4	97	.786	1.340	9.54	20.04	.309	0.99	0.88	−0.04	1.58	2.
	B	9.2	100	.786	1.326	9.64	20.26	.311	0.97	0.96	−0.02	1.41	2.
	C	8.9	102	.778	1.338	9.56	20.06	.300	0.93	1.04	−0.01	1.38	2.
1:1½:3	0	9.6	97	.780	1.391	12.25	25.75	.358	0.79	1.00	+0.13	1.44	3.
	5L	10.1	97	.776	1.414	12.05	25.32	.350	0.83	1.04	+0.03	1.48	3.
	3⅓K	10.1	98	.774	1.411	12.08	25.39	.348	0.83	1.16	−0.10	1.48	3.
	1⅔C	10.5	99	.772	1.418	12.01	25.26	.345	0.87	1.08	−0.15	1.66	3.
	10L	10.7	96	.772	1.439	11.85	24.90	.342	0.88	1.00	−0.09	1.61	3.
	6⅔K	10.6	100	.768	1.430	11.93	25.12	.340	0.87	1.12	−0.11	1.65	3.
	3⅓C	11.4	98	.756	1.446	11.79	24.77	.326	0.94	1.12	−0.21	1.84	3.
	A	9.9	97	.780	1.412	12.06	25.36	.353	0.82	0.83	−0.15	1.02	2.
	B	9.4	98	.780	1.388	12.27	25.80	.357	0.78	0.88	−0.27	0.90	2.
	C	9.1	98	.774	1.400	12.18	25.58	.350	0.76	0.92	−0.19	0.91	2.
Average of all Results		9.8	98	.780	1.334	8.76	18.41	.281	1.165	1.01	−0.09	2.10	3.
Average of Fine Sand Results		9.8	98	778	1.336	8.75	18.39		1.16	1.04	−0.08	1.98	3.
Average of Coarse Sand Results		9.8	99	.780	1.333	8.77	18.44		1.17	0.98	−0.10	2.17	3.

Workability—The arrangement of Table I is such as to make possible a ready comparison of the effects of corresponding quantities of the powdered admixtures, hydrated lime, kaolin and celite. Since the percentages of these materials in each group are in the proportions of 3: 2: 1 respectively (based on the assumption that these proportions would show about the same effect on workability), it is interesting to see how the workability figures bear out this assumption. The irregularity in the workability figures through the successive groups make it difficult to decide how the admixtures compare in the proportions given, and the tendency is best shown by tabulating all the figures as in Table II.

TABLE II.—RELATIVE EFFECT OF HYDRATED LIME, KAOLIN AND CELITE ON WORKABILITY OF CONCRETE MIXTURES.

Per cent Admixture.	Workability Figures.				
	1:1⅓:3	1:2:4	1:2½:5	1:3:6	Average.
5 Lime	33	103	167	207	
10 Lime	31	81	125	185	
15 Lime	..	..	129	171	
Average	32	92	140	188	113
3⅓ Kaolin	31	100	150	180	
6⅔ Kaolin	27	79	130	199	
10 Kaolin	..	..	105	154	
Average	29	90	128	178	106
1⅔ Celite	38	98	167	227	
3⅓ Celite	18	74	114	192	
5 Celite	..	..	116	160	
Average	28	86	132	193	110

The average workability figures for the lime, kaolin and celite given in Table II confirm the earlier conclusion that three parts of lime, two parts of kaolin and one part of celite have practically the same effect upon workability.

The effect of the admixtures on workability are also shown in Fig. 3. In Fig. 3 the workability figures are plotted to logarithmic scale as ordinates, the different groups of mixtures shown in Table I being arranged approximately in the order of increasing workability. The average values of workability for each group are connected by broken lines. These broken lines show the trend of the effects of the admixtures quite distinctly, but the departure from smooth curves shows the uncertainty of the penetration test and the need for a test of greater precision. We may call attention,

however, to the effects of the three accelerators, which are always in the order C, A, B, with marked differences between them; also to the fact that in these tests the high percentages of admixtures raise the workability of any given basic mixture nearly to that of the next richer mixture; and finally to the comparatively erratic results in the 1 : 1½ : 3 mixtures, which under ordinary conditions would not generally require the use of admixtures.

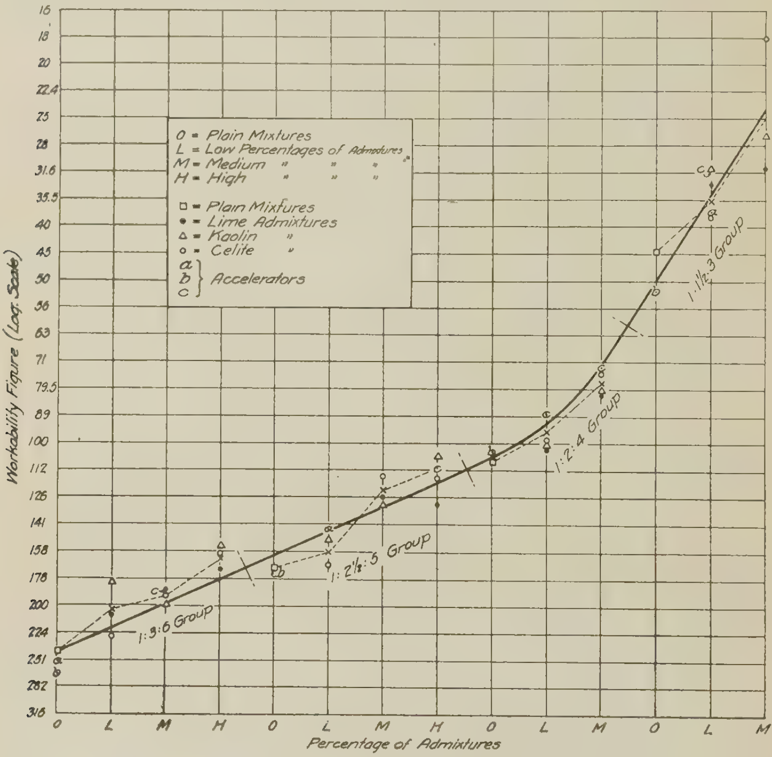


FIG. 3.—EFFECTS OF ADMIXTURES ON WORKABILITY.

Quantity of Mixing Water—Bearing in mind that the flow figures were designed to be the same, we find that comparatively small variations in the amount of mixing water occurred throughout the entire range of mixtures, rich, lean, harsh or plastic. In all cases, however, the celite admixtures required appreciably more water than the others, while accelerator C required the least amount of water. With these two exceptions, and noting a small increase in the water toward the richer end of the series, the variation in water content is not much greater than in the flow figures themselves.

Water-Cement Ratio and Cement-Space Ratio—These factors will be discussed later in their relations to the strength variations.

Density—The uniformity in density throughout the entire range of concrete is remarkable. With one exception all densities lie between the values .768 and .787, and yet the effect of the admixtures is plainly indicated. In each of the several groups lime gives the highest density and celite the lowest. There is also a similar difference noted in the accelerator groups, *A* giving the highest density and *C* the lowest. These differences, small though they are, seem to depend mainly on the water content, with little or no relation to the proportions of the solid constituents in the various mixtures.

Yield—The yield values increase fairly regularly from the lean mixtures to the rich over a range of about 0.17. Accelerator *B* produces the lowest yield in any given mixture; *A* and *C* produce about the same yield, slightly higher than that of the plain mixture. The powdered admixtures always increase the yield, kaolin generally giving the lowest and celite the highest, in any given group.

Plastic Shrinkage or Settlement in the Forms—The figures are expressed in percentages of the volume of the freshly placed concrete. It will be noted that the shrinkage in all cases is practically 1 per cent, with a tendency to be slightly higher for the richer mixtures.

Amount of Cement in Finished Concrete—The figures in column 7 are expressed as per cent of cement by absolute volume in the finished concrete. The figures in column 8 are obtained by multiplying those in column 7 by a factor which gives the per cent of cement *in bulk*. For example, the plain 1: 3: 6 mixture contains 14.02 per cent of a sack of cement in each cubic foot of finished concrete. The figures vary directly with the richness of the mixture, and inversely as the yield values.

Loss in Weight During 28 Days of Storage under Different Conditions—The specimens stored in the damp closet show as a rule a slight gain in weight. Those exposed to lower humidities show a higher loss for the lean mixtures than for the rich mixtures. In general the average loss in weight for the specimens stored out of doors was a little more than half that for those stored in the laboratory. Particularly in the case of the specimens stored in the laboratory the effects of the admixtures are quite plainly shown in the several groups. The loss increases in the order of lime, kaolin, celite; for the accelerators in the order C, B, A, in all cases.

Strength—The strength results are interesting in that they show quite remarkable uniformity in each of the four main groups, regardless of admixtures or storage conditions. Only in the 1: 1½: 3 group is there serious loss of strength due to the admixtures. Averages of all the strengths in each of the three last columns of Table I give the values 1774, 1619, and 1558, which are to each other as 100, 91 and 88, respectively. That is, if

the strength of the specimens stored in the damp closet is taken as the basis of comparison the strength of the specimens stored out of doors shows a difference of only 9 per cent, and that of those stored in the laboratory a difference of only 12 per cent, on the average. That these differences are not greater is due in part to the high humidity which prevails in Washington during the summer, and in part to the fact that the strength of dry or partially dry concrete is considerably greater than that of wet concrete.

We may examine the effects of the admixtures on strength by means of the following table, in which the average strengths of each of the small groups in Table I is expressed as a percentage of the strength of the corresponding plain specimens. For convenience we will refer to the small groups as "Low," "Medium," "High" and "Accelerators" in Table III, the significance of these terms being the same as in Fig. 3.

TABLE III.—EFFECTS OF ADMIXTURES ON THE STRENGTH OF CONCRETE

Relative Strength.

Mix.	Admixture.	Damp Closet.	Outside.	Laboratory.	Average.
1:3:6	0	100	100	100	100
	Low	103	105	102	103
	Medium	104	106	99	103
	High	105	105	99	103
	Accelerators	104	114	107	108
1:2½:5	0	100	100	100	100
	Low	100	98	99	99
	Medium	101	98	96	98
	High	97	94	93	95
	Accelerators	105	107	107	106
1:2:4	0	100	100	100	100
	Low	98	104	103	102
	Medium	94	95	96	95
	Accelerators	108	113	118	113
1:1½:3	0	100	100	100	100
	Low	94	93	96	94
	Medium	87	86	86	86
	Accelerators	103	112	113	109

From Table III we see that except in the 1 : 1½ : 3 mixtures, the maximum loss in strength caused by the largest amounts of admixtures is 7 per cent, and that the maximum average loss for the three storage conditions is only 5 per cent. In general the leaner concretes stored in the damp closet benefit from the powdered admixtures, whereas those stored in the laboratory lose slightly. This seems to indicate that the powdered admixtures are of no assistance under dry curing conditions. On the other hand the accelerators give increased strength under all conditions, and a relatively greater increase under dry storage.

It is also interesting to see how the admixtures compare with each other in their effect upon strength. In Table IV the average strengths obtained from each admixture in each basic mixture are expressed as a percentage of the strength of the corresponding plain specimens.

While the differences in the effects of the admixtures on strength are small, they are unmistakable, and in general vary with the differences in

TABLE IV.—COMPARATIVE EFFECTS OF DIFFERENT ADMIXTURES ON THE STRENGTH OF CONCRETE.

Relative Strength.

Mix.	Admixture.	Damp Closet.	Outside.	Inside.	Average.
1:3:6	0	100	100	100	100
	Lime	107	109	105	107
	Kaolin	102	105	98	108
	Celite	103	102	98	101
	A	100	108	102	103
	B	101	114	108	108
	C	111	119	111	114
1:2½:5	0	100	100	100	100
	Lime	102	100	101	101
	Kaolin	99	96	94	96
	Celite	97	93	93	94
	A	99	103	104	102
	B	106	107	108	107
	C	111	111	108	110
1:2:4	0	100	100	100	100
	Lime	98	103	101	101
	Kaolin	95	100	101	99
	Celite	95	95	97	96
	A	103	107	110	107
	B	111	112	123	115
	C	108	118	119	115
1:1½:3	0	100	100	100	100
	Lime	90	91	93	91
	Kaolin	91	89	93	91
	Celite	89	88	87	88
	A	90	103	103	99
	B	109	116	117	114
	C	110	117	118	115

water content for which they themselves are responsible, if we accept the flow test as the criterion for "consistency." Of the powdered admixtures, hydrated lime in the proportions used is most favorable to strength; of the accelerators, C gives uniformly the highest strength.

It is finally a matter of interest to see how closely the concrete strengths obtained in this series fall in line with those indicated by the "water-cement ratio" and the "cement-space ratio." In Fig. 4, the com-

pressive strengths of the cylinders stored in the damp closet only are plotted against these two factors. The curves are drawn designedly through the values obtained from the four basic mixtures, first because each of these points is derived from an average of ten tests, second because we are particularly interested in noting the effects of the admixtures and the deviations which they may produce from the basic curves.

In examining Fig. 4 we see that all the strength values lie somewhere near the basic curves, but we note certain regularities in the departure from these curves that are significant. For example, in the strength-water

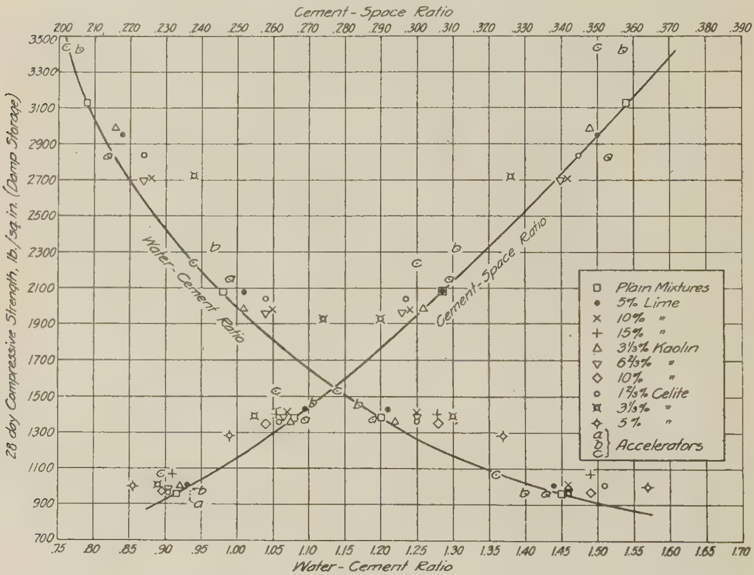


FIG. 4.—RELATION BETWEEN COMPRESSIVE STRENGTH OF CONCRETE AND (a) WATER-CEMENT RATIO AND (b) CEMENT-SPACE RATIO.

relation all of the powdered admixtures cause a displacement of strength values to the right of the basic curve; in other words, the strengths are always higher than the water-cement ratios indicate. In the case of the highest celite admixtures the strengths are very nearly 20 per cent higher. We note also that the effects of the admixtures on strength are relatively much less in the leaner mixtures, the general line of each group of points flattening out as we follow down the basic curve.

Somewhat similar departures from the basic cement-space ratio curve are noted, most of the points also lying above the curve, and indicating as before a somewhat higher strength than the theory permits. If we omit from consideration the concretes containing the accelerators, it is possible that the average deviation of the strengths from the space-ratio curve is

slightly less than from the water-ratio curve, but there is not much choice. One peculiarity is noted in the fact that the strengths of concrete containing accelerator *C* exactly coincide with the water-ratio curve, whereas they are far above the space-ratio curve. This is apparently related in some way to the foaming caused by accelerator *C*, which undoubtedly accounts for its rather remarkable effect on workability, and yet allows it to produce the highest average strength obtained in the series.

SUMMARY.

With reference to the questions on page 5, we may summarize the most important results of this investigation as follows:

(1) The workability of any concrete mixture is about equally benefited by one part of celite, two parts of kaolin, or three parts of hydrated lime, such as used in these tests, if the consistency as measured by the flow table is kept constant. Under the same conditions plain calcium chloride did not improve the workability, whereas of the two proprietary compounds used one was found to give a very appreciable improvement and the other quite a marked improvement.

(2) The powdered admixtures in the proportions used did not seriously affect the strength of 1 : 2 : 4 or leaner concretes; in fact the 1 : 3 : 6 concretes were benefited by the admixtures in all cases. Considerable reduction in strength resulted from the larger proportions of the admixtures in 1 : 1½ : 3 concrete. With reference to actual concreting operations these conclusions are conservative, for it was observed that as the workability of a mixture was improved, the tendency was to use less mixing water than was required to maintain the prescribed flow. In other words, the more plastic mixtures seemed over wet in comparison with the others, and more water was used in them than appeared to be necessary or would probably be used under job conditions to obtain a desired "consistency." The results of the tests reported herewith indicate that a judicious use of admixtures in concretes which need improvement in respect to workability will not materially impair the strength. The chloride admixtures without exception produced a considerable increase in strength.

(3) The slight tendency toward reduction in strength of concretes containing the powdered admixtures was if anything more pronounced under dry or partially dry curing conditions than under damp storage conditions. On the other hand the chloride admixtures were relatively of more benefit under dry conditions.

(4) No appreciable effect on the shrinkage of the plastic concrete was noted from the use of admixtures.

(5) The increased yield of concrete containing powdered admixtures is in favor of the latter, and in many cases is probably sufficient to entirely

offset the cost of the admixture. The net saving from the use of admixtures is not in cost of materials, but rather in labor cost and greater assurance of uniformity in the finished product.

(6) The voids in concrete are affected only slightly by admixtures, apparently reflecting the effect of the admixtures on the gauging water. If admixtures necessitate the use of more water for a given consistency, the voids will be increased proportionately, other conditions being the same.

(7) Powdered admixtures do not seem to prevent loss of water from concrete by evaporation under dry or partially dry curing conditions, but chloride admixtures reduce the loss under these conditions. Corresponding effects upon the compressive strength were noted above.

(8) The authors believe that the data presented herewith justify a more general consideration of the proper use of admixtures. Better and more economical concrete must result from anything that improves the plasticity or workability of the mixture, provided no deleterious effect is also involved. It has been shown that small proportions of powdered admixtures do not detract appreciably from the strength of the concrete except in the rich mixtures, where the need for and the values of such admixtures disappear. On the other hand the improvement in workability that can be brought about by the judicious use of admixtures is considerable, and this must tend to reduce costs and improve the uniformity of the finished product, under given labor conditions.

As a guide to "judicious use" of powdered admixtures, the tests indicate that the amount of an admixture to be recommended is an inverse function of the richness of the mixture. For the materials used in this investigation, the maximum percentage of celite is approximately the number of parts of coarse aggregate (relative to the cement) used in the mixture; the maximum percentage of kaolin is about twice the number of parts of the coarse aggregate, and the maximum percentage of hydrated lime is about three times this figure. For example, in a 1:2:4 mixture the maximum percentages of admixtures to be recommended are about 4 per cent of celite, 8 per cent of kaolin, and 12 per cent of hydrated lime, by weight of the cement. The improvement in workability which is effected by these maximum additions is about that which should be expected from a 25 per cent increase in the cement content. Larger amounts of the admixtures than those specified above may be used in cases where the highest obtainable strength is not of prime importance.

With reference to chloride admixtures, there is no doubt that these are coming rapidly into general use, and there is no count against them, so far as we are aware, in plain concretes and mortars. Those preparations which also improve the workability of cement mixtures have an added value, over and above that of their accelerating properties.

SUPPLEMENTARY STATEMENT IN REGARD TO THE MEASUREMENT OF WORKABILITY.

As stated in the foregoing discussion, the penetration test for workability has served a useful purpose, but it has a number of defects. In the first place it is an indirect test measuring only the comparative segregation and lubrication in different mixtures, in an arbitrary and meaningless system of units. Second, it is lacking in desirable precision, and the average results from a large number of tests must be taken in order to insure a reasonably accurate rating for a given mixture. Third, it is not applicable to a study of the effects of different coarse aggregates, except within a narrow range of gradation.

It is possible that the penetration test may be improved in such manner as to eliminate some of the above mentioned objections, but in the light of further experience with the test, we are inclined to attack the problem anew along entirely different lines. It now appears that a more general solution of the problem should be sought by the application of methods used in studying the properties of plastics modified in such manner as to deal with relatively large additions of coarse, non-plastic materials. It is obviously impossible to force mortar and concrete mixtures through capillary tubes and thus determine the relations between pressure and flow, and it appears to be impracticable to make such determinations merely by enlarging any of the existing types of viscocimeters. The nature of the mixtures with which we are concerned seems to require some sort of a stirring device in which the relations between power and speed can be determined, and this leads to the thought that a modification of the ordinary concrete mixer may be the apparatus which will afford a general solution of the problem. It should be possible to separate the power required for lifting the concrete and overcoming the friction losses from that required to drive the blades of a stirrer through the mass, and if this can be done, we may be able to deduce therefrom the characteristics of any given mixture. The incentive for getting a satisfactory general solution for this problem lies in the fact that some very important uses of Portland cement depend for their success upon maintaining a definite degree of plasticity while other properties are being changed. We believe also that specifications for important concrete construction will some day contain requirements for workability—not, however, the quality indicated merely by the slump or the flow test.

DISCUSSION.

G. W. Hutchinson.—To the parties engaged in the study of concrete and concrete proportioning, the data so ably presented by Messrs. Pearson and Hitchcock appear as an opening of which advantage may be taken to learn more of the fundamental properties of concrete. To the engineer and user of concrete it presents a line of reasoning, which, if taken seriously, will be a great contributing factor in the control of field concrete with respect to ease of placement and uniformity of product, both of which are of supreme importance in economic structures. A discussion of a paper of this kind, if it is to be of value, will require data on tests as well as observations. Inasmuch as I have been interested in the use of admixtures for a number of years, I have taken the opportunity to present such data as has been secured in the laboratory as well as in the field. Its presentation will include a discussion of the advantages of the seeming necessities of admixtures from the field as well as the laboratory standpoint.

C. M. Upham, State Highway Engineer of North Carolina, was probably responsible for the first use of admixtures on large scale concrete road operations. These were used in the Coleman duPont road in Delaware, in 1915, and specified when the Delaware State Highway Department first specifications were written. Considerable laboratory work was carried on at that time, and it was found that admixtures most available had practically little effect on the resulting strength of the laboratory concrete. The results in general showed a slight increase. When used in small proportions it contributed a very desirable element in workability and seemed to assist in the finishing of the concrete road. Later we found that with coarser sands there was more to be gained along workability lines than with a more finely graded type of fine aggregate. In writing specifications for the State Highway Commission of North Carolina, while we did not specify the use of admixtures, we included in the specifications a paragraph which would allow us to take advantage of their qualities, when certain materials indicated the need of improved workability. Workability under these circumstances was measured by judgment alone, inasmuch as we had no way of determining this property by measure.

In this connection it must be realized that the engineer engaged in public work is expected to have his decisions governed by fact rather than by opinion, and inasmuch as the benefits derived were measured by observation alone, the use of them was temporarily tabled with slight exception in North Carolina until such time as more study could be given and more information be made available. This action was also prompted by the fact that the fine aggregate encountered in this locality was of a generally fine gradation, which in itself assisted in the workability, and probably most important was the fact that sufficient knowledge of the great variation in field concrete was not available at the time to justify the great amount of investigation necessary in work of this kind.

The invention of the device as described in the paper under discussion is felt to be the greatest contribution to this line of study. Although, as stated in the paper, it is not as yet a perfect means, I believe that it will be acknowledged to be far better than any other method yet discovered. The preliminary story told by this instrument seems to answer to a great degree the question of what workability really is, and it was responsible for the outlining of an extensive program to study the economic value of available admixtures in North Carolina. This work is being carried on by the Division of Tests and Investigations of the North Carolina State Highway Commission. Inasmuch as investigations of this nature must be confined to the materials most available, a survey indicated that there were three to be considered.

First was a material known as talc dust, which is manufactured in North Carolina. This material was relatively cheap and readily available in quantities. Second was hydrated lime which was also available in quantity, and then due to exceptional results secured by the Bureau of Standards along workability lines, came the selection of celite, which is the trade name for a certain type of specially prepared diatomaceous earth, coming from Lompoe, California.

Laboratory investigation was carried on consisting of compressive tests of 4000 6 x 12 in. concrete cylinders cured under different storage conditions, and containing different cement contents. These were undertaken along lines practically parallel to the investigation undertaken by Messrs. Pearson and Hitchcock.

In addition, field work is also being carried on to determine the economy of admixtures from the viewpoint of construction. A discussion of the investigations can probably be best presented by offering the results as an attempt to answer the questions found in the paper under discussion.

The work carried on relates principally to questions Nos, 2, 3, 5, 6, 7 and 8, and the discussion will be from the practical side supported by the laboratory results.

Question Two.—Is the reduction of strength of concrete containing admixtures in small proportions serious?

This is with reference to the importance of a reduction in strength of concrete containing admixtures. Our experience is that in the nominal proportion of concrete with nominal amounts of certain admixtures there is no reduction in strength, but in practically all cases a slight increase is obtained and this increase is relatively greater as the cement content is decreased. Any disagreement of test results regarding strength can probably be attributed to the various gradations of aggregates used in the different investigations. It is safe to say, however, that with any gradation of aggregate allowable for use, any decrease in strength by the use of approved admixtures, would not be appreciable. In the proportions recommended for use, the test results should generally fall within the limits of test error in laboratory concrete, which will account for seeming inconsistencies in the detail of tests of this nature. It will be noted that the

figures given both in the original paper and this discussion do not take into consideration the effect of admixtures on the yield or volume of concrete secured. While this subject will be taken up under Question Five, it should be stated here that an increase in yield when admixtures are used will decrease the relative cement content of the mixture when compared with basic mixtures, or plain concrete. Inasmuch as cement is the only binding ingredient in concrete, this is an important factor, when comparison is made on the basis of equal cement content per volume of concrete. When this factor is considered, the concrete containing the admixtures will, in general, increase in strength ratio proportional to their effect on the bulking or yield of the concrete containing them.

The laboratory data in Table 1 consists of tests of about two thousand 6 x 12 in. cylinders containing three types of admixtures, four lots of cement, and cured under wet and dry conditions. Inasmuch as the error of tests and other accidental variations affect the detail results, possibly outside of the limits of the effect of admixtures, all results are averaged in Table 1. Each result is the average of twenty specimens tested at the age of twenty-eight days.

TABLE 1.—EFFECT OF ADMIXTURES ON THE RELATIVE COMPRESSIVE STRENGTH OF CONCRETE.

Admixture.	Cement, per cent.				Average.
	15	20	25	30	
Plain Concrete	100	100	100	100	100
2½ per cent Hydrated Lime	103	100	98	98	100
5 per cent Hydrated Lime	104	98	96	95	98
7½ per cent Hydrated Lime	106	102	96	97	100
4 per cent Talc Dust	101	98	98	96	98
8 per cent Talc Dust	101	96	93	94	96
1½ per cent Celite	101	107	100	101	102
3 per cent Celite	104	107	94	93	100
4½ per cent Celite	103	107	95	90	99
6 per cent Celite	108	105	97	...	103
8 per cent Celite	109	...	...	...	109
Average (Admixture)	104	102	96	96	...

NOTE.—Cement content measured as volume of dry cement per unit volume of concrete. For arbitrary proportions interpolations may be made as follows:

	Per Cent.
1:1½:3	28
1:2:4	22
1:2½:5	18
1:3:6	15

All admixtures are proportioned by percentage of weight of the cement content. A 3 per cent addition to the 20 per cent mix would therefore require a greater quantity of admixture than a 3 per cent addition to the 15 per cent mix, etc.

Relative unit weight of Materials:

Talc Dust	55
Hydrated Lime	40
Celite	10

Question Three.—Is the effect of admixtures on strength of concrete more favorable under average job conditions than under standard laboratory conditions?

The analysis of field concrete is a difficult proposition, especially when data from numerous projects are included. It is possible to arrive at conclusions of a certain nature when data from a single structure or project are analyzed, as the range of variation is generally smaller than when compared with data from a number of structures or projects. This, of course, is due to the greater number of variables entering into the latter which should allow broader and more accurate conclusions than would be available in the individual investigation, although at best they can be but general in scope. Research of this nature must be carried on in such manner as to be of benefit to the whole instead of being capable of application to but one situation.

It is questionable if the testing of specimens cast in the field will be of much assistance in determining, within reasonable limits, the quality or uniformity of the concrete in place. The specimens themselves might give indication of a uniform concrete of a given strength, but when compared with the results of tests of cores drilled from the same batch from which they were made a great variation may be secured.

The difference in compressive strength between drilled road specimens, corrected for size and the standard 6 x 12 in. cylinder from the same concrete, has been investigated by the North Carolina State Highway Commission on thirty projects. In addition to this, sufficient materials were taken from each of the individual batches of dry materials, composing the field concrete, to allow specimens to be made and stored under standard laboratory conditions.

All molded specimens, both in the field and laboratory, were made by experienced laboratory operators, and the general recommended practice in manipulation was followed.

The results given are the average of four specimens of the field concrete and five of the laboratory concrete.

Sufficient material was taken after the concrete had been placed to make four of these specimens from different parts of the batch. The location of this batch, with reference to the road, was then marked for the core drill. The field specimens were cured beside the road for twenty-one days and then packed in damp sand and shipped to the laboratory to test at the end of twenty-eight days.

Table 2 gives the result of this investigation. It should be noted that the relative values are obtained from the result of one correction for size of specimen in the molded field concrete, and this same correction in addition to one for age in the case of the field core. The field concrete averaged three and one-half months old when tested. The variables, due to average correction, however, would be relatively small when compared with the variation in the results actually obtained.

TABLE 2.—SHOWING COMPARISON OF TESTS OF CONCRETE SPECIMENS DRILLED FROM FIELD CONCRETE AND SPECIMENS MOLDED IN THE FIELD AND LABORATORY.

No.	Proj.	Mix.	Field Cast Specimen.	Field Drilled Specimen.	Laboratory Cast Specimen.	Ratio, Laboratory Cast Specimen to Field Drilled Specimen.
1	446	1:2½:5	100	65	116	1.78
2	577	1:2:4	100	68	62	0.91
3	630–B	1:2½:5	100	79	54	0.68
4	342	1:2½:5	100	86	69	0.80
5	494	1:2½:5	100	94	102	1.09
6	460	1:2½:5	100	96	106	1.10
7	822	1:2½:5	100	100	81	0.81
8	535	1:2:4	100	101	95	0.94
9	412	1:2:4	100	107	69	0.65
10	386–7	1:1½:3	100	107	83	0.77
11	643	1:2½:5	100	111	78	0.70
12	671	1:2½:5	100	115	100	0.87
13	428	1:2:4	100	116	101	0.87
14	741	1:2:4	100	117	92	0.79
15	632	1:1½:3	100	122	95	0.78
16	588	1:2:4	100	123	103	0.84
17	473	1:2:4	100	123	80	0.65
18	743	1:2½:5	100	124	80	0.65
19	614	1:2:4	100	133	89	0.67
20	429	1:2:4	100	134	123	0.92
21	484	1:2:4	100	136	97	0.71
22	463	1:2:4	100	146	126	0.86
23	528	1:2:4	100	154	109	0.71
24	765	1:2:4	100	155	54	0.35
25	679	1:2½:3	100	157	129	0.82
26	742	1:2:4	100	170	109	0.64
27	670	1:2:4	100	173	112	0.65
28	790	1:2:4	100	193	...	....
29	785	1:2:4	100	198	90	0.45
30	629	1:2:4	100	204	84	0.42
Maximum	....	....	100	204	129	1.78
Average	....	....	100	127	93	0.65
Minimum	....	....	100	65	54	0.35

NOTE.—Field cast specimens tested at age of 28 days and corrected for height. Field drilled specimens corrected for height and age for 28 day results. Laboratory cast specimens tested at 28 days (standard height.

It must be realized that the strength of concrete, regarded on the basis of usual laboratory comparison, is the least of the problem to be solved. A few per cent variation from the average strength whether an increase or decrease is negligible. The main purpose is to control the uniformity by reducing the variable caused by manipulation. An efficient admixture of any kind which will reduce the amount of labor necessary for manipulation will not only accomplish this result, but will also reduce the cost. There is no economic advantage in design for high average strength, if accompanied by great variation throughout the structure, but there is a decided one in concrete of any designed strength possessing uniformity. A typical case of the advantage of uniformity is derived from an analysis of results taken from field tests of concrete pipe from twelve different plants extending over a period of about a year. Due to the wall thickness of the smaller sizes of pipe being greater than would normally be necessary to withstand the specified strength test, there were but fifteen

per cent rejection of the smallest size, and about twenty-five per cent rejection of the next size larger. The size next larger in diameter, although having a wall thickness sufficient to withstand the crushing load required by the specifications, when made from a practical mix, did not have as great a margin above them as the smaller sizes and resulted in nearly sixty per cent rejection. The same class of concrete was placed in all sizes of pipe but the specifications regarding wall thickness being somewhat unbalanced, placed a premium on uniformity in the larger sizes of pipe, with the result that over one-half of the product was condemned principally due to non-uniformity of the concrete. Single line reinforcement was used in this case mentioned, and although no data are available in double line work, it is reasonable to assume that with a double line required, the demand for greater workability and uniformity would be more apparent.

Another question to be raised on the subject of strength is with reference to materials and age. There is a change in the relative strength between gradations of aggregates as the actual strength increases with age. There is a similar effect encountered when changes are made in the cement content. These changes in relative strength, when caused by lack of sufficient control in the proportioning of the dry materials, furnish a variable in relative strength at the different ages which is considerably in excess of the effect of any approved admixtures. Considerable investigation of this matter is under way at the present time and if early tests are indicative of the later results, attention should be directed to the determination of the proper sizes of coarse aggregate for field use. It is felt to be but a matter of time when all large concreting operations will require the proportioning of coarse and fine aggregate on a weight basis. This will eliminate considerable of the variation in the concrete and make the advantages of admixtures more apparent.

It is extremely doubtful if the true effect of admixtures on the workability, strength and uniformity can be determined in the laboratory. By this I mean that there are many conditions to be met in the field that are not encountered in laboratory work. Laboratory manipulation will counteract certain adverse conditions which the admixture is expected to correct in the field, and thus the value of the admixture is not fully brought out in laboratory tests. The virtue of the admixture lies in its ability to assist in making a more uniform and workable concrete in the field. The uniformity is of decided advantage from a structural viewpoint, and the workability has its value with reference to the economy of placing the concrete at decreased cost, or producing a more uniform finished structure, at the same cost. The relative value of different types of admixtures will be dependent upon their ability to accomplish the desired purpose at the lowest cost. It must be understood that the use of admixtures of any kind cannot be regarded as a panacea for all causes of the variation in field concrete. With such causes divided into two classes, first the variation encountered in materials, and second the variation due to manipulation; their ability to correct is only with reference to the latter.

It is stated that the importance of the question of workability has been grossly underestimated, probably for the reason that there has been no answer for it. I would like to add another reason for this to the effect that it is doubtful if but few realize the great actual variation in field concrete. If more data on this subject were available there would be considerably more agitation to develop means for its control.

A comprehensive study of detail regarding the reasons for the effect of admixtures under field conditions requires a considerable amount of time and study, and due to the work of this nature in North Carolina not having reached the stage of final analysis, the field data are not available at

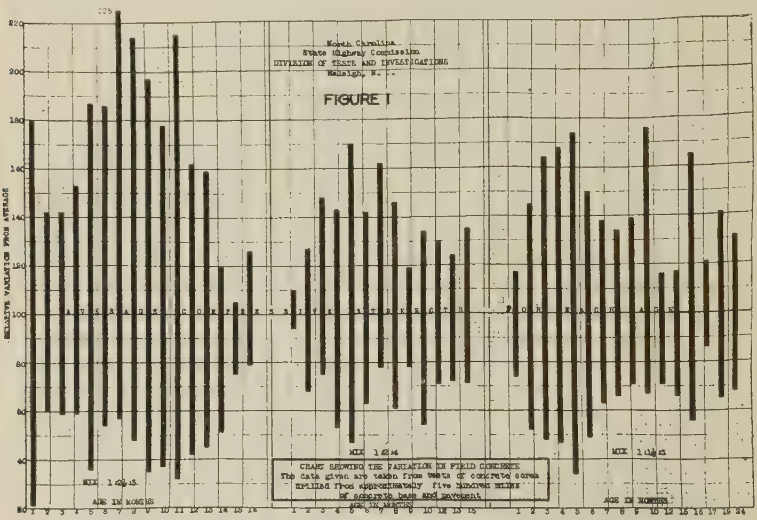


FIG. 1.—CHART SHOWING THE VARIATION IN FIELD CONCRETE.

Data are taken from tests of concrete cores drilled from approximately 500 miles of concrete base and pavements.

the present time. Work in connection with the use of celite as an admixture has been placed and more is being carried on at present. This material was used entirely in the field on account of its great effect on the workability and its apparent economy. The data given represent an attempt to show the necessity of control in the mixing and placing of concrete.

The variables include coarse aggregate from over twenty deposits, fine aggregate from thirty deposits, and cement from over fifteen different mills, as well as practically every method of handling materials acceptable to construction of a large scale.

Fig. 1 shows the problem in detail and includes the result of all variables found in tests of concrete cores drilled from about five hundred miles of concrete road and base work laid under these conditions. These tests

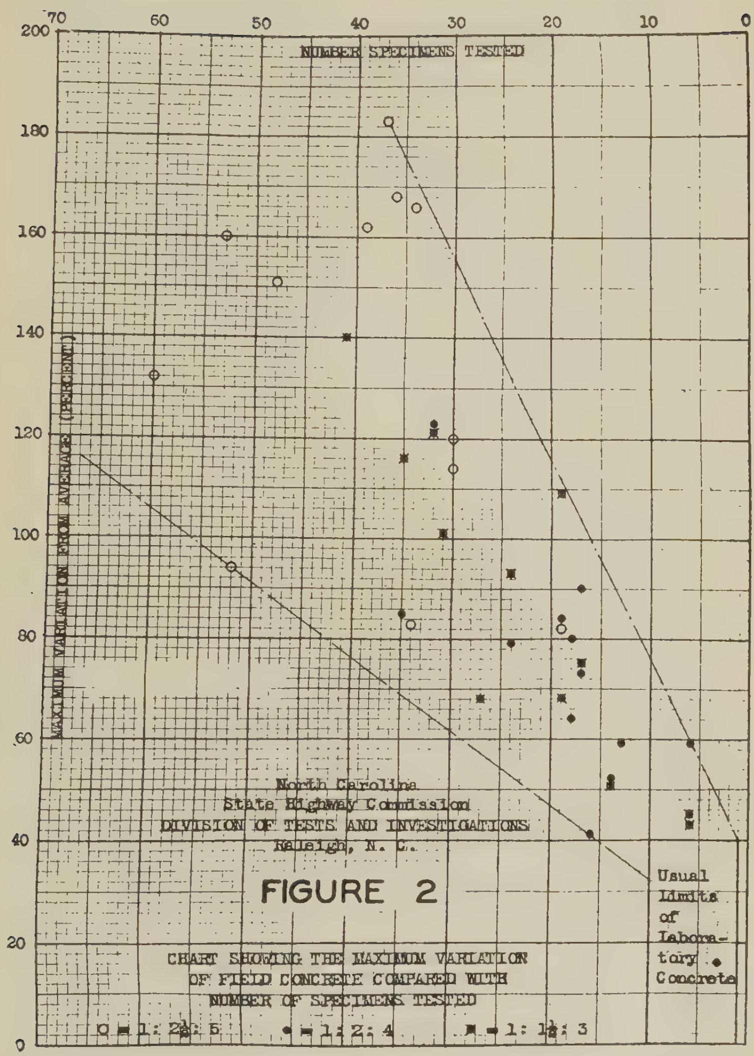


FIG. 2.—CHART SHOWING MAXIMUM VARIATION IN FIELD CONCRETE COMPARED WITH NUMBER OF SPECIMENS TESTED.

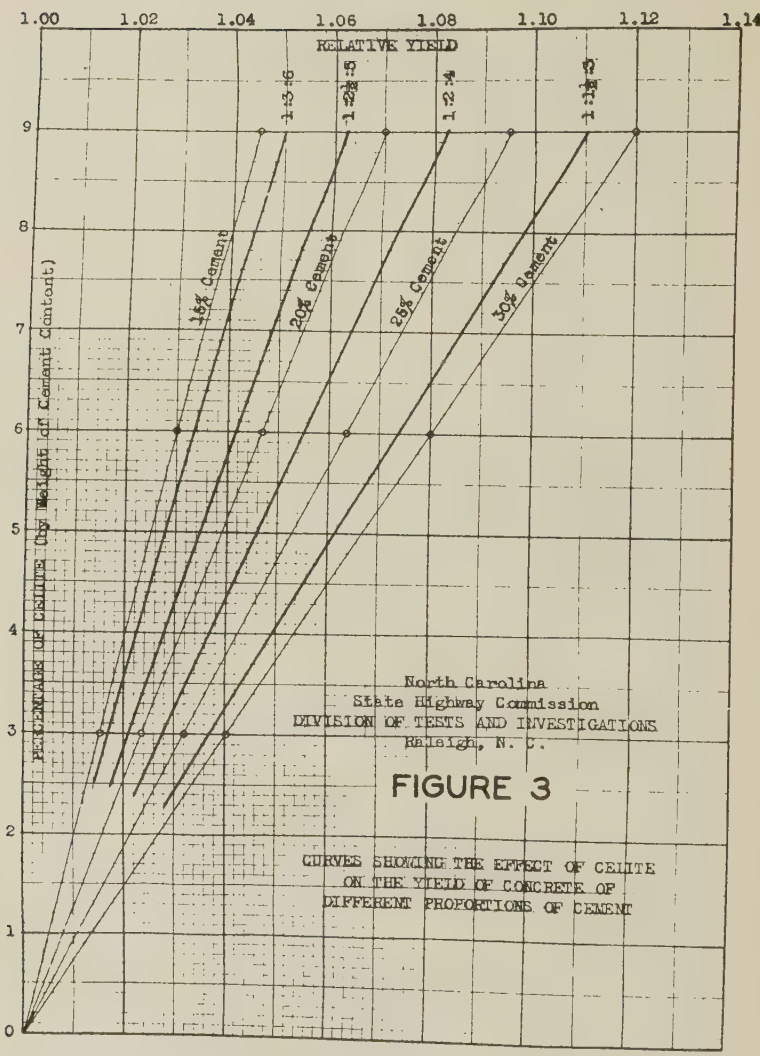


FIG. 3.—CURVE SHOWING EFFECT OF CELITE ON YIELD OF CONCRETE OF DIFFERENT PROPORTIONS OF CEMENT.

are separated on the basis of age of specimen and by different proportions of mixture. It will be noted that there is an apparent decrease in variation as the concrete gets older. This is due largely to the fact that a lesser number of specimens were available for test in the older concrete which would decrease the amount of data affecting the range of maximum variation.

Inasmuch as we are only concerned at the present time with the variation due to manipulation, an attempt is made to analyze the data on this basis. The data are platted in Fig. 2 to show the relation between the maximum variation and the number of specimens tested. In this work the average of all concrete tests of a given age, in months, is taken as a basis for compilation. The variation possible to encounter in the general run of laboratory testing is also shown in Fig. 2 for comparison. It will be noted that the variation found in field concrete is greater, as would be expected, when the number of specimens tested is increased, and that it is considerably more than that found in laboratory concrete. The points brought out indicate that:

1. Any means for promoting workability or uniformity in concrete should function much more efficiently in the field than in the laboratory. This is due to the range of variation in the field being so much greater.

2. If these means are accomplished by the use of powdered admixtures, the effect of such admixtures on the strength would be negligible when compared with the variation actually encountered in the field.

Question Five.—Do admixtures increase the "Yield" of concrete sufficiently to effect an appreciable saving in cost?

Due to the same workability results being secured by Messrs. Pearson and Hitchcock with considerable less celite than with other admixtures, this material was selected for further investigation, and the accompanying figure indicates the result of yield tests made to determine the bulking effect of this material. It will be noted in Fig. 3 that a direct relation between the amount added and the yield of the resulting concrete was secured with this admixture, and while the cost of the material in question is probably as high in North Carolina as in any part of the country, its effect on the yield, as shown, is such that under conditions it can be used at no additional expense whatever.

There is no doubt but that the bulking effect of admixtures is due partly to the water it carries into the mass. The effect of this water on the strength of concrete is not detrimental as shown by the strength tests and further indicated by Fig. 4 in the paper under discussion, where the water cement ratio is platted against the compressive strength.

Question Six.—To what extent are the voids or the density of concrete affected by the use of admixtures?

It will be noted that in the paper presented that the densities of the concrete measured vary but little. This is another point where field con-

crete varies from laboratory concrete, as the methods of mixing and placing, etc., of the concrete in the laboratory allow considerably more control of this factor than in the field, and therefore would not allow the admixture to function as effectively as could be expected in the field. Field concrete should be mixed with as little water as possible so that the consistency will be as dry as can reasonably be used in placing the mixture properly. It is obvious that the more workable the concrete is, the dryer it may be in consistency and at the same time be placed with the same amount of labor. Using the figures as given in the paper under discussion, it is possible to increase the workability of the concrete about thirty per cent accompanied by an increase in the water cement ratio of about eight per cent with practically identical strengths. In concrete road work it has been found not only desirable, but under certain finishing conditions, necessary to mix concrete of a drier consistency when the admixture was used.

Question Seven.—Do admixtures tend to prevent loss of water from the concrete by evaporation under dry or partially dry storage conditions?

It will be noted that the actual loss of water was greater with the specimens containing admixtures. This statement, however, is without reference to the fact that more water was used in the mixtures containing admixtures.

The computation for loss of water, to be absolutely correct, should take into consideration the amount actually used in the mixture as well as the amount of shrinkage during setting. With these factors considered, it would be a question if there actually was a relative loss, or if in some cases a relative gain might not be observed.

Question Eight.—Is the use of admixtures considered advantageous and advisable, all things considered?

This question is in general more or less of a summary of the advantages and possible disadvantages due to the use of approved admixtures. In an analysis of this nature, the concrete must be pictured in place. The laboratory is only of value when its determination can be practically applied. There are many troubles apparent in the fabrication of field concrete, however, which can only be counteracted in the field.

The paper under discussion brings out what is to my mind a very important factor in field concrete. We have learned that wet concrete is inferior to that relatively drier but at the same time workable. This, to date has been measured by consistency alone. It can now be seen that workability and consistency are two entirely different things, and the former is far the most important in field work, as the latter is but a secondary matter when practical application is concerned.

The use of admixtures should not only be considered with reference to the elimination of segregation when the mass is being placed but should also be given credit for functioning while the materials are in the mixing drum. It should be realized that the usual mixing time is by no means sufficient to make a uniform concrete, and that any process that will

decrease friction between the particles during this period will allow a greater dispersion within the given period.

It has been stated that the effect on strength in the laboratory is negligible and that such conditions as found in the laboratory will have a tendency to eliminate certain field problems the powdered admixture is intended to solve. Possibly the best example of this might be found in tests of concrete pipe. Tests of laboratory made concrete pipe showed an increase in crushing strength of about five per cent when three per cent of celite was added, but when the same amount was added in the commercial product, a fifteen per cent increase in strength and greater uniformity was obtained. The same amount of admixture, however, was used in the same proportions of concrete (1: 1½: 3) but placed in the standard 6 x 12 in. cylinder, which required a considerably smaller quantity of concrete, showed no increase. This would indicate that the size of the structure is to be considered. The general run of admixtures show no indication of possessing cementing qualities, so that under no circumstances should they be substituted for cement. Their value should be in producing workability and uniformity alone, and their cost should be judged only after the advantage along these lines has been considered. The admixture itself must also be considered as it does not follow that any material in powdered form will be suitable.

When the problems encountered in field concrete are considered, it is surprising how little advantage has been taken of the vast amount of laboratory work which has been carried on. This may be due to the fact that considerable of the data available are not applicable to field results and partly due to inability to get the same results in the field as were secured in the laboratory. A large part, however, must be due to ignorance of the variation of the product, or if such knowledge is available, to ignorance of how to control it.

DUFF A. ABRAMS (*by letter*).—The paper by Messrs. Pearson and Hitchcock is an important contribution to our knowledge of the effect of inert powdered admixtures in concrete. The new method of measuring workability of concrete is an interesting modification of the flow test. Certain defects in the test are pointed out and subjects for future investigation are indicated. It would be of interest to study this test in concrete made of crushed aggregates.

While the new test for workability is a step in advance for research work, it is the writer's belief that the slump test made in the manner recommended by the American Society for Testing Materials has a decided place, especially as a control test on the work where more refined methods are unnecessary. Too much accuracy must not be expected in such a test; the main thing is to see that concrete of a slump of 8 or 10 in. is not used when the quality of concrete required in the work is such that a slump of 3 or 4 in. should be used.

As soon as engineers, owners and contractors fully realize that proper control of quantity of mixing water pays big dividends and that using an

excess of mixing water produces exactly the same effect on the concrete as cutting down the cement in the batch, there will be a marked improvement in common practice in this report.

Compression tests were made on four concretes using three admixtures in the percentages shown below:

Concrete Mix	Lime	Kaolin	Celite
1 : 1½ : 3	5, 10	3.3, 6.7	1.7, 3.3
1 : 2 : 4	do.	do.	do.
1 : 2½ : 5	5, 10, 15	3.3, 6.7, 10	1.7, 3.3, 5
1 : 3 : 6	do.	do.	do.

The author points out that the effectiveness of the same quantities by weight of hydrated lime, kaolin, and celite is approximately proportional to the numbers 1, 2, and 3. It seems to the writer that these materials can most conveniently be studied when expressed in terms of the volume of cement. The weight of the lime, as reported in a paper by the same authors before the 1923 Convention of the American Society for Testing Materials*, was about 32 lb. per cu. ft., kaolin 34.5, and celite about 10.5. For the same methods of test the unit weight of the cement was 89 lb. per cu. ft. Therefore, if the percentages by weight are expressed as percentages by volume, we find that in the case of hydrated lime and celite, 14, 28 and 42 per cent of admixture by volume of the cement were used in the 1 : 3 : 6 and 1 : 2½ : 5 concrete mixtures, and 14 and 28 per cent for the 1 : 2 : 4 and 1 : 1½ : 3. The corresponding percentages by volume for kaolin, calculated in the same way, were about two-thirds of these values. Thus the authors' data show that equal *volumes* of celite and hydrated lime had approximately the same effect on the workability of the concrete. In the case of kaolin only about two-thirds as much was used to produce the same effect.

Tests of the effect of powdered admixtures on the strength of concrete carried out in the Structural Materials Research Laboratory, Lewis Institute, Chicago, have shown that the hydrated lime and other inert powdered materials reduced the strength nearly in proportion to the quantity of admixtures. For the usual concrete mixtures the reduction in strength varied from 0.25 to 0.75 per cent for each 1 per cent of admixture *by volume* of cement.* These tests, using the slump method, failed to show any increased workability by the use of powdered admixtures. In any discussion of this subject the effect of admixtures on workability is

*A Penetration Test for the Workability of Concrete Mixtures with Particular Reference to the Effects of Certain Powdered Admixtures, by J. C. Pearson and F. A. Hitchcock, Proc. Am. Soc. for Testing Mat., vol. 23, 1923.

*See "Effect of Hydrated Lime and Other Powdered Admixtures in Concrete," Proc. Am. Soc. Testing Materials, Part II, 1920; also Proc. 1921. Reprinted as Bulletin 8, Structural Materials Research Laboratory, Lewis Institute, Chicago.

vital and consequently the method of measuring workability is of paramount importance.

It appears that certain important relations shown by these tests were not sufficiently emphasized by the authors. We might add the following to the questions propounded:

Question (a) Does portland cement affect the workability of concrete in the same manner as the other powdered material used?

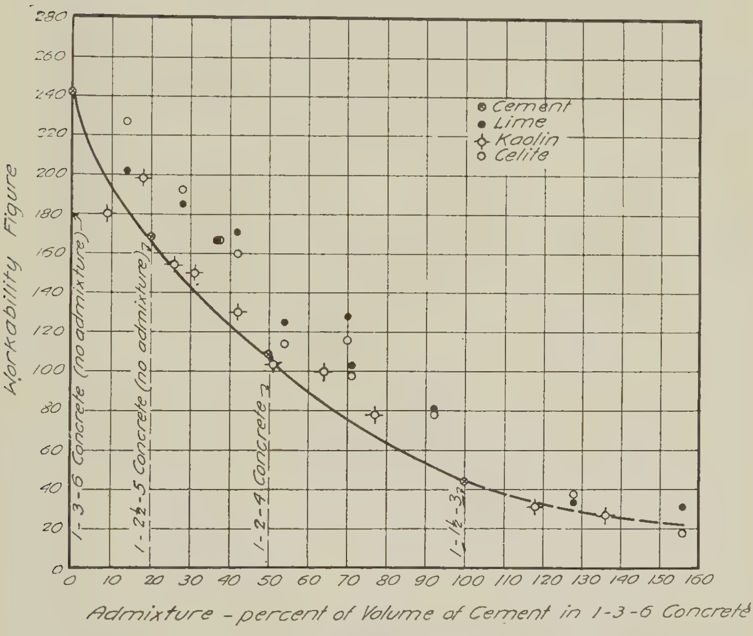


FIG. A.—EFFECT OF CEMENT AND POWDERED ADMIXTURES ON THE WORKABILITY OF CONCRETE.

Data from "Economic Value of Admixtures," by Pearson and Hitchcock.
Comparisons made for concrete containing same volume of powdered material (cement plus admixture) expressed in terms of volume of cement in 1 : 3 : 6 concrete.
Workability measured by penetration method.

Question (b) Without regard to concrete strength, what material produces the desired workability at the least cost?

Question (c) For a given concrete strength, with what material can a desired workability be obtained at the least cost?

Question (a).—The authors point out that the workability is improved by the addition of 25 per cent cement to about the same extent as by addition of 4 per cent by weight of celite, 8 per cent of kaolin or 12 per cent of lime. It is significant that these four percentages *expressed by*

weight represent approximately the *same volumes* of the material, and that on a volumetric basis, portland cement is about as effective in increasing workability as the other powdered admixtures.

It will be of interest to make a further study of the effectiveness and economy of cement as compared with the other powdered admixtures. To

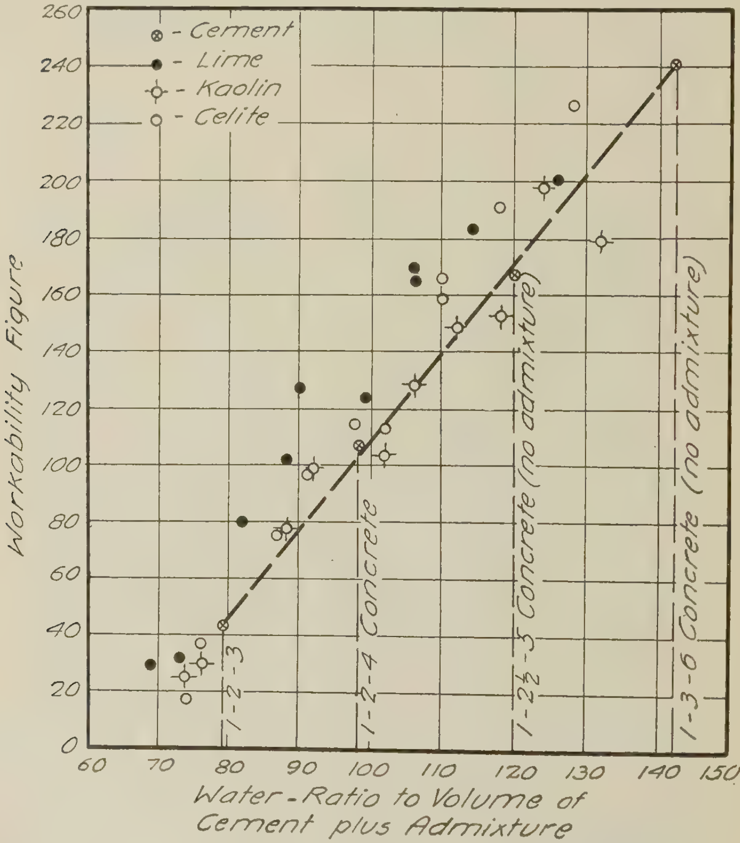


FIG. B.—EFFECT OF CEMENT AND POWDERED ADMIXTURES ON THE WORKABILITY OF CONCRETE.

Data from "Economic Value of Admixtures."
Comparisons made for concrete containing same quantity of mixing water, expressed in terms of volume of cement plus admixture.
Workability measured by penetration method.

facilitate these comparisons, the 1 : 3 : 6 concrete, without admixtures, has been taken as a basis, and additional volumes of the admixtures (including cement) are expressed as percentages of the volume of the *cement* in 1 : 3 : 6 concrete. For example, a 1 : 1½ : 3 concrete is equivalent to a 1 : 3 : 6 mix with 100 per cent admixture of cement; similarly the

1 : 1½ : 3 concrete with an admixture of 14 per cent of powdered materials by volume is treated as a 1 : 3 : 6 concrete with 12.8 per cent of admixture.

Fig. A gives the relation between the "workability figure" and the percentage of admixtures on this basis. The curve was drawn through the values for concrete containing cement only. Since only one of the

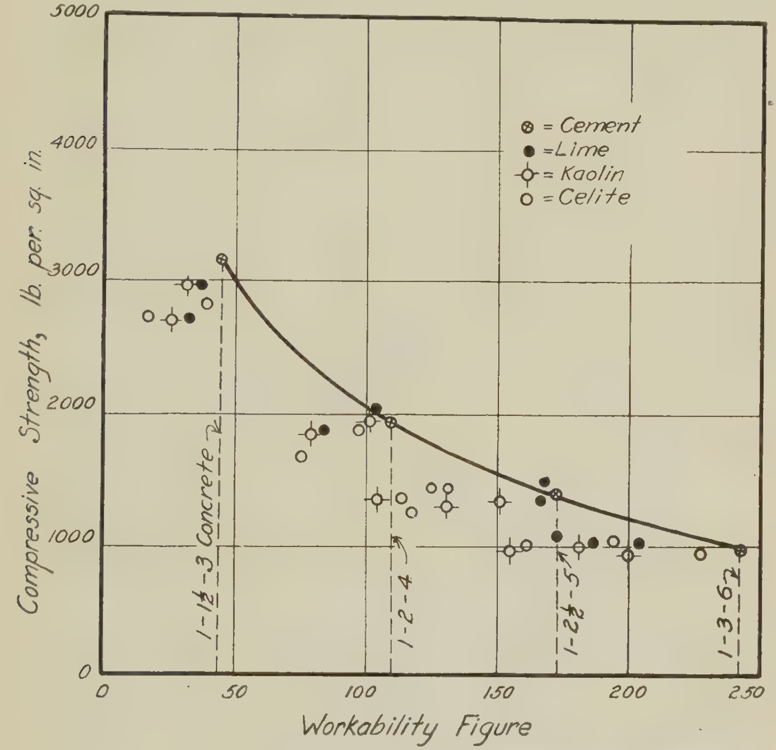


FIG. C.—RELATION OF WORKABILITY AND STRENGTH OF CONCRETE.

Data from "Economic Value of Admixtures."
Compression tests of 6 by 12-in. concrete cylinders cured in "damp closet."
Workability measured by penetration method.

values for the powdered admixtures fall to the left and below the curve for "plain" concrete, it is evident that for a given volume of admixtures, cement was most effective in increasing workability.

Our Fig. A is based on the same data as Fig. 1 in the paper. It is difficult to see the justification for the continuous curve in Fig. 1. Such a curve gives an erroneous impression, since there is no direct relation between the adjacent groups of points (for different mixes); there are in reality four unrelated segments.

While Fig. A gives a fair representation of the data, the cement is not compared on the same basis as the other admixtures, since approximately the same water-cement-ratio was used for a given mix, regardless of the quantity of admixture. A more accurate comparison is to express the water-ratio in terms of the volume of cement and admixture. In Fig. B this method has been adopted. The water-ratio is expressed in terms of the volume of cement plus admixture. The curve is drawn through the

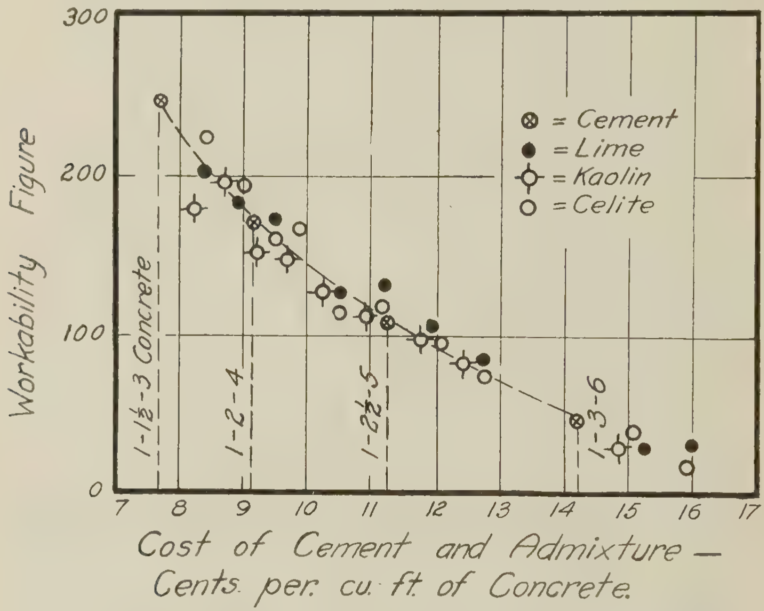


FIG. D.—COST OF CEMENT AND ADMIXTURES IN CONCRETE OF DIFFERENT WORKABILITY.

Data from "Economic Value of Admixtures."
Comparisons made for equal workabilities; strength variable.
Workability measured by penetration method.
Compare Fig. E and F.

points for the plain concrete. This seems permissible since the powdered admixtures have about the same normal consistency, by volume, as cement when determined by the Vicat needle. This diagram shows that essentially the same degrees of workability were obtained for the different materials (cement, lime, kaolin, and celite) when compared in this manner. While the points cover a fairly wide band, no decisive advantage is shown for any one material.

Question (b). Fig. C shows the relation between workability and strength of concrete cured in a moist room, and shows that except in two instances, for equal workability the plain concrete has considerably higher

strength than that containing admixtures. Concrete stored out-doors and in the Laboratory were not studied in this way, as the authors state in Conclusion 3 "The slight tendency toward reduction in strength of concrete containing powdered admixtures, was, if anything, more pronounced under dry and partially dry curing conditions, than under damp storage conditions."

Fig. D shows the relation between workability and the cost of the cement plus admixture, per cu. ft. of concrete. The following prices were

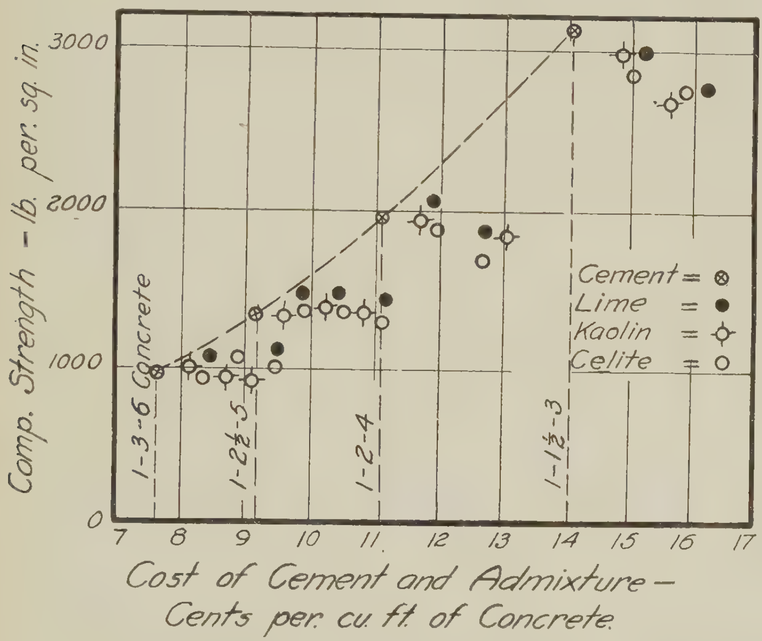


FIG. E.—COST OF CEMENT AND ADMIXTURES IN CONCRETE OF DIFFERENT STRENGTHS.

Data from "Economic Value of Admixtures."
Compression tests of 6 by 12-in. concrete cylinders cured in "damp closet."
Compare Fig. D and F.

assumed for carload lots in Chicago: cement, $2.20 per bbl. (about $12 per ton of 2000 lb.); lime, $20 per ton; kaolin $25 per ton; celite $60 per ton. In estimating the amount of material required per cu. ft. of concrete, the percentages of cement given in Table 1 of the paper were used. The cement was assumed to weigh 94 lb. per cu. ft. The curve shows that the cost of the cement required to give a certain degree of workability is about the same as for the other admixtures, and that no advantage is shown for one over the other. Any desired degree of workability can

be secured as cheaply with cement as with any of the admixtures investigated.

Question (c). Fig. E shows the relative cost of cement plus admixture in a cu. ft. of concrete compared on the basis of the strength of cylinders stored in the damp closet and tested at 28 days. The curve is drawn through the points for "plain" concrete. The points for the admixtures are to the right and below the plain concrete curve, showing that for equal strength plain concrete costs from 1 to 2 cents less per cu. ft. than concrete with the admixtures.

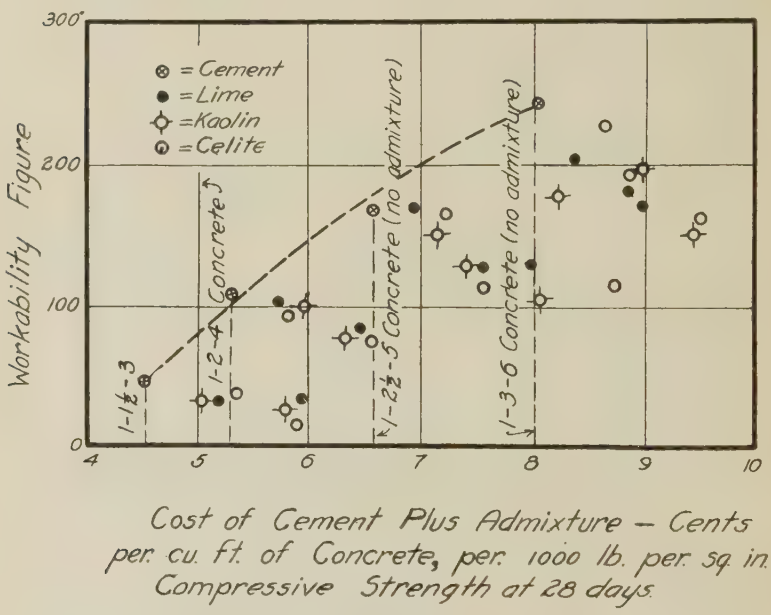


FIG. F.—COST OF CEMENT AND ADMIXTURES IN CONCRETE OF DIFFERENT WORKABILITIES BASED ON CONCRETE OF SAME STRENGTH.

Data from "Economic Value of Admixtures."
Compare Fig. D and E.

Fig. F combines the information in Fig. D and E and shows the relation between workability and cost for equal strengths. The cost of cement plus admixture per cubic foot of concrete was divided by the strength (in 1000 lb. per sq. in.) of specimens stored in the damp closet and plotted against the workability of the concrete. The curve was drawn through the points for "plain" concrete. It will be seen that for a given workability the cost of cement plus admixture is less for "plain" concrete than for concrete containing the other admixtures.

The above diagrams show that concrete of the same workability can be obtained by using additional cement as well as by the use of admixtures, and at no greater cost. The additional cement will, at the same time, increase the strength of the concrete about 1 to 1½ per cent for each 1 per cent used; that is, the addition of 10 per cent of cement to a concrete of the usual mixtures will increase its compressive strength approximately 10 to 15 per cent. When the cost per volume per unit of compressive strength is considered, it is much more economical to obtain the desired workability by means of cement than by any of the admixtures investigated. The cost of the concrete made with the admixtures does not include the added cost of handling a separate material in comparatively small quantities.

It is interesting to note that the water-ratio strength curve from the tests of Messrs. Pearson and Hitchcock (Fig. 4) is almost identical with the curve of this type which appears in the writer's paper on "Design of Concrete Mixtures."

WHAT WE MAY EXPECT TO DO WITH ALUMINATE CEMENT THAT WE CANNOT DO WITH PORTLAND CEMENT, BASED ON COSTS AND RESULTS OBTAINED ABROAD.

By Henry S. Spackman.*

My first impulse on reading hurriedly Mr. Whipple's letter asking me, as the American discoverer and patentee of aluminate cement to talk to you of these cements at this meeting, was to decline. It seemed too much like asking a fond father to express publicly his dreams of his son's future when the boy was still in swaddling clothes. But on re-reading I noticed the saving words at the end of the title, "based on the costs and results obtained abroad." This let me out. It was not of the infant American industry I was to talk but of his big cousin, that lusty, thriving, still growing young giant, the French aluminate cement industry, which has established a right to demand your serious consideration of its product by twelve years' use in all fields of construction and four years' use under the trying conditions of modern warfare. I am not going to try and tell you what aluminate cement is from the chemical standpoint or to explain the scientific reasons for its high strengths at short periods and its resistance to attack by sea water. Instead, I shall confine myself to statements of what it has done and will do in actual work.

Aluminate cement has two exceptional qualities that differentiate it sharply from all other hydraulic binders. These are the rapidity with which it attains maximum strength and its resistance to chemical attack by sea water and waters containing sulphates of the alkaline earths in solution. We will consider first the remarkable high strengths of aluminate cement at short time periods, that is, at 24 hours, 3 days and 7 days. In this development of high strength at early periods aluminate cement is as superior to portland cement as portland cement is to natural cement, or natural cement is, in turn, to lime. The difference in the rate of hardening between aluminate cement and other hydraulic cements is as great relatively as the difference in time of traveling between the ox-cart that brought our forefathers to Chicago and the Pennsylvania Limited that brought me. Aluminate cement concrete mixed one cement, two sand, four stone or gravel, will give at 24 hours a resistance to compressive stress of 3,000 lb. per sq. in.; at 3 days, 4,000 lb. per sq. in.; at 7 days, 5,000 lb. I am not referring now to strengths developed in laboratory tests, but to the strengths you can safely count on getting in the work using average aggregate and workmanship. Tests were made on 6-in. cubes.

What does such early strength mean to the worker in concrete? It means that a concrete road can be opened to traffic 48 hours after pouring. For example, at Le Teil, France, a section of concrete road poured between 8 p. m. Saturday and midnight was opened to traffic early Monday morn-

*Consulting Engineer, Philadelphia, Pa.

ing and carried a five-ton tractor hauling heavy loads without marking the concrete. At Lausanne, Switzerland, a bridge was opened to traffic 48 hours after the concrete was poured. A 12-ton tractor passed over the bridge the first day without the slightest injury. These early strengths also mean that in reinforced-concrete work you can pour a story one day and use the forms again three days after. This was done in the construction of a round house for locomotives at Betheune, France, for the Railroad Company of the North. L. Peulabeuf, the contractor, used aluminate cements. The design called for the construction of 150 concrete chimneys, each 23 ft. high, and weighing 4,500 lb., carried by a reinforced-concrete roof and girders which had a 20 ft. span. The forms for both roof and chimneys were removed 48 hours after pouring. Using portland cement the forms and supports had to remain in place 20 days. It is proper, however, to state that French portland cements are slower hardening than American.

In reporting on the use of aluminate cement for the construction of a reinforced-concrete building at Chateau Roux, Indre, France, M. Blanchett stated that the forms were removed in 48 hours from the beams and 3 days from the girders, and that a saving of two-thirds of the form lumber that would have been required for doing the work with portland cement was made.

Aluminate cement also makes possible the use of reinforced-concrete in the structural alterations of occupied buildings. In such work aluminate cement can be used with the same felicity as structural steel. Perret Frères, of Paris, were entrusted by the Bank La Societiee Marsiellaise de Credit with the alteration of a five-story building on Rue Auber, Paris. It was proposed to substitute for a maze of small rooms and passages on the ground and second floors, the large reception halls and offices of a modern bank. In visualizing the problem, you must realize that the partitions were all masonry extending up from the cellar and carried the superimposed load of the three upper floors. To make the task more difficult, it was stipulated that the work must be carried on without interrupting the labor of the office force working in these two stories and without inconvenience to the tenants who occupied the three upper floors. Structural steel columns and girders could not be used because it would have been impossible to get them into the building and put them into place without interfering with the occupancy of the two lower floors. Reinforced portland cement concrete could not be used because it would have necessitated the leaving of the forms and supports in place for several weeks in the office space required for the employees. Recourse was had to aluminate cement, which met the requirements of this difficult situation in every particular. Forms were removed in 48 hours and by reason of its greater strength a considerable reduction was made in the area of columns and depth of girders.

Actual use in winter work has shown that the hardening of aluminate cement is less affected by low temperatures than is the hardening of portland cement concrete. The French claim that provided the concrete is kept

from freezing when it is being put into the forms and for two or three hours after, the development of strength will not be retarded by cold, the heat given off in setting being sufficient to prevent freezing. Whether this would be true in the severe cold of our northern winter is doubtful to me, but the ability to harden at lower temperatures than portland is a great advantage for late fall and early spring work, as it will obviate the annoying condition that most workers in concrete have met with from time to time by having concrete, though unfrozen, lie inert in the forms for days because of the low temperature of the concrete, even though the temperature of the air is quite high. The manufacturers of concrete products find the rapid hardening of aluminate cement concrete of great advantage. It reduces the number of moulds required, does away with expensive storage sheds for curing the products and the tying up of capital in products under process of hardening. The concrete products can be shipped 24 hours after casting. It also permits the manufacture in the field and the almost immediate putting into use of such products as water pipe, reinforced piles, poles for support, electric wires, etc. Reinforced-concrete piles can be driven three days after casting and water pipe put under heavy pressure in 48 hours.

It will probably be interesting to you at this point for me to discuss the relative difference in cost between aluminate and portland cement concrete. According to the best information I can obtain, aluminate cement in France sells at the mills for between two and a half and three times the price of portland cement. This means, allowing on the average $1.00 per barrel for handling, that is, railroad freights, hauling, etc., that aluminate cements cost delivered on the work from two to two and a half times as much as portland cement. For example, if portland cement cost $2.00 F. O. B. mill, and aluminate cement $6.00, with the addition of a handling cost of $1.00, the cost delivered on the job will be $3.00 for portland cement and $7.00 for aluminate cement, which is two and one-third times the cost of portland. At first thought, such a difference may seem prohibitive to you except in special cases where the time element or resistance to chemical attack makes the question of cost negligible. Such, however, has not proved to be the case in France. The best evidence of this is that it is today impossible to buy aluminate cement anywhere in Europe for immediate delivery. All the mills manufacturing aluminate cement are over sold. It is more difficult to obtain exact cost data than information regarding the behavior on the work. Therefore, I can quote only general statements.

Mr. Blanchet writes: "One sees that without mentioning the quickness of the process, that the economy realized in the forms makes up and more for the higher price of aluminate cement."

Mr. Louis Peulabeuf writes: "I believe that aluminate cement will be the cement of the future for reinforced-concrete work, when its price will permit one to employ it in a more general fashion. Nevertheless, for the present in spite of its high price if one takes into account the economy in

forms that it permits one to make, there will be found a large compensation for its cost."

Edwin C. Eckel, who made a study of the aluminate cement industry in France during the past summer, writes in December *Concrete:* "The apparent disadvantage, its necessarily high cost per barrel, is only apparent, for aluminate cement whatever its cost per barrel makes a cubic yard of concrete harden in place at a lower cost than any other cement owing to the economies possible in proportions, labor, forms and time."

Following the same line of thought as Eckel, Prof. Paris, of Lausanne, has made a series of calculations which show that at current prices in Switzerland, the cost per 100 lb. per sq. in. resistance at 28 days in concrete is less for aluminate cements than for portland. At shorter periods, three days for example, the cost of 100 lb. per sq. in. of resistance to compression with aluminate cement is one-third that of portland.

Aluminate cement is no longer regarded as new or experimental in France. It is considered competitive with portland in much work and, in addition, has a recognized though constantly expanding field of its own, in which the specification of aluminate cements for work is considered almost mandatory. They are used in the largest work, for example, the tunnel at Brauss required some 75,000 bbl. and practically all the cement work used in Paris for street work is now aluminate cement.

Aluminate cement, however, has a quality even more valuable than the development of high strength at early periods, that is, resistance to chemical attack of waters carrying sulphate salts in solution. Extended laboratory research and results of use have proved beyond question of doubt that portland cement is not stable in sea water. Though many portland cement concrete structures exposed to the action of sea water have been and are giving good service, this resistance to the destructive action of sea water is not due to any particular virtue of the cement used, but to the protective action of the aggregate which prevents the sea water from coming in contact with the cement. Any portland cement concrete, exposed to sea water, sufficiently porous to allow the water to penetrate, it will inevitably be destroyed by chemical action, and what is true of sea water is equally true of ground water impregnated with sulphate salts of the alkaline earths. Therefore the importance of resistance to chemical attack is not confined solely to marine structures. Indeed, the ground water of many of our large cities is contaminated by sulphur. In Philadelphia, this is particularly noticeable. I have known the sulphur content of the ground water to prevent concrete from hardening in deep foundations for two or three weeks. What the ultimate effect of this sulphur on the durability of the foundation will be is not difficult to predict.

In Europe, owing to the character of their coasts, there is much port and harbor work which requires a greater use of concrete in sea water than is common here. For this reason the failure of cement in sea water has been given more careful study than with us. As early as 1853, on account of the numerous reports of the deterioration of marine struc-

tures, the French Society for the Encouragement of National Industry offered a reward for the best study on the cause of failure of cement in marine work, and for the past seventy-five years European engineers, chemists and cement manufacturers have been seeking to find a cement that would be indecomposible in sea water, and it was the continued search for such a cement that led ultimately to the development of the aluminate cement industry in France.

It may be well at this point to touch upon the genesis of aluminate cements. The story of their discovery is that of two investigators, widely separated by distance, each ignorant of the other's work, and each starting with a slightly different aim in view, reaching in the end the same conclusion and results. I refer to the work of Monsieur Jules Bied and myself.* Mr. Bied, the Chemical Director of the La Farge Company, to whom the development of the aluminate cement industry in France is due, started his research with the specific object of finding a low lime hydraulic cement that would be resistant to the attack of sea water. Whether it was higher testing than portland cement or not was immaterial, as was the question of rapid hardening. The *sine qua non* was that it must be resistant to the attack of sea waters. On the other hand, my object was to develop a cement that would be quicker hardening, stronger and more durable under ordinary conditions than portland cement, without specific regard to its behavior in sea water. In aluminate cement we both found our desire realized, but it was the resistance of aluminate cement to sea water, not its high strength at early periods, that led to the development of the aluminate cement industry in France, and I do not think the especial value these early strengths gave it for use in general construction was realized at first in France. At all events, I can find no record of the use of aluminate cements solely because of its higher early strength prior to the use for artillery foundations, etc., during the war. Before that aluminate cements seemed to have been used only where subject to chemical attack.

To return to our subject, the deterioration of portland cement in sea water is due to two causes, one physical, the other chemical. The physical decomposition is caused by the crystallization in the pores of the concrete of the salts dissolved in the water. This crystallization is produced when the concrete is exposed to alternate wetting and drying. It is mechanically similar to the effect of freezing and can be guarded against by using dense, impermeable mortars. Chemical decomposition is caused by the sulphur attacking the lime or high lime compounds in the hydrated cement, forming basic sulpho aluminates. The very bulky crystals of this salt which develop in the pores and interstices of the concrete act as innumerable wedges forcing it apart. If the current of the sea water percolating

---

* It was with great sorrow that I learned since the writing of this paper of Mr. Bied's recent death, as during my two years' service in France as Commanding Officer of Cement Section, A. E. F., we saw much of each other and became strong personal friends, and have since co-operated and exchanged information concerning aluminate cement.

through the concrete is sufficiently rapid, the sulpho aluminates may be washed out and the action continued until the cement gangue be completely decomposed to a sandy mass. Both the physical and chemical action is accentuated by wave action, and in many cases of failure both causes of decomposition are present, each accentuating and intensifying the action of the other.

Immediately upon Bied's announcement of aluminate cement and its production in sufficient quantity, it was subjected to the most rigid tests in both governmental and private laboratories in Europe and under conditions of actual work. These tests proved, as would be expected from its chemical composition, its resistance to the attack of sea water. The test of the Paris Lyon Meddeterannie Railroad Company, begun in 1916, made necessary by the destruction by sulphated waters of the Brauss tunnel, show that aluminate cements resist perfectly the attack of water impregnated with sulphate of lime, even when a large part of the sand is replaced by anhydrate. As a result of these tests, aluminate cement was used in the reconstruction of the tunnel and is now specified by the railroad company for all concrete work exposed to sulphate carrying waters.

Dr. Jeaneret made very severe comparative tests between portland and aluminate cement, specimens were made one part cement, two parts sand and one part gypsum. These were rammed into molds and then immersed in water. The test specimens made of portland cement began to disintegrate shortly after two months' time. After three months the rapidity of disintegration was increased. At the end of four months the aluminate cements showed absolutely no trace of destruction and gave compression strengths almost three times greater than was obtained from the two portland cements tested.

Candelot writes: "Whatever the explanation, the long experience we have now had with aluminate cement has shown that its resistance to sea water action is perfect not only in the laboratories, but in extensive practical use. * * * The aluminate cement therefore resolves all question as to concrete construction either in sea water or in alkaline soils."

Other advantages not so immediately obvious as high strength at early periods and resistance to the attack of sea water are claimed for aluminate cement. It is stated that the concretes made from aluminate cement are more elastic and have higher tensile strength than those made from portland. These characteristics combined with the high resistance to compressive stress make possible a marked reduction both in the area of reinforced-concrete members and in the amount of reinforcing still required. On one piece of work, a tobacco manufacturing plant involving the use of about 150 yards of reinforced-concrete, a saving was made through reduction in area of 14 per cent of the amount of aggregate and cement required and of 7 per cent in the amount of reinforcing steel. The design and calculation for the reinforced-concrete work on this job were made by Mr. Guillaume, the consulting engineer of the Societee Central de Travaux Public. Such a reduction in area more than compensates

for the extra cost of the cement and leaves the indirect gain, such as saving in time, amount of form lumber, etc., clear profit.

The second of these advantages is that the hardening of aluminate cement concrete is not materially affected by low temperatures.

The third is that with any given fine aggregate, aluminate cement used in the same proportions will give a denser, more impermeable and less absorbent mortar than portland cement. This is an important factor in connection with resistance to the physical attack by sea water on concrete structures.

Aluminate cement mortars are more plastic than portland cement and do not shrink in setting. These characteristics combined with quick hardening and impermeability, make aluminate cement particularly adapted for outside plaster.

In my talk today I have purposely confined myself to a bare recital of information that has come to me from France. I have also limited the discussion to fields in which aluminate cement is directly competitive with portland and have cited in support of each major claim actual work giving the names of the place where used and of the user. I could multiply these citations indefinitely, but believe I have said enough to convince you that using aluminate cement it is possible to build concrete marine structures with the positive assurance that they will be immune from the insidious chemical attack of salt water and that you can put plain or reinforced concrete structures into service and subject them to the full load for which they are designed in from 48 to 72 hours after pouring.

As to how and to what extent aluminate cements will affect the concrete industry in this country for the future, you gentlemen are in a better position to express an opinion than I. But, in closing, I will make this prophecy,—as portland cement has been the magic and reinforcement the great romance of concrete during the past twenty years, so aluminate cement will be the fairy wand whose use will make a reality of your dream of lighter, more elastic, more beautiful and more enduring concrete structures for the future.

DISCUSSION.

P. H. Bates.—Though Col. Spackman's paper is extremely interesting, I am sorry that I cannot be particularly enthused with the prophecy which forms the conclusion of the paper. Acknowledging that these alumina cements are superior in many ways to portland cement when measured in terms of the latter, yet there are a number of questions which must be considered other than simple strength alone. I believe it were desirable to not anticipate too much, but bear in mind all the conditions obtaining abroad, where these cements have so far alone been produced.

At the present time these cements are manufactured alone by Ancienne Société J. & A. Pavin de LaFarge. This is a French company which is generally known in this country as the LaFarge Cement Co., and one of its products has enjoyed almost constant use in this country for a number of years, namely, LaFarge White Portland Cement. But the alumina cement, which is the cement referred to by Col. Spackman, and this LaFarge White Portland Cement are only two of a large number of cements which this company manufactures. In addition to these, it produces several varieties of portland cement designed to have certain, but different, setting and hardening qualities; a number of natural cements, hydraulic limes, and slag-portland cement. It produces all of these for a definite purpose, and the consuming public in France apparently knows sufficient about these different cements to use them properly. This company is not advocating or believing that the alumina cements will be the cement of the future, but rather, one of a number of cements. This is well illustrated by the fact that the Paris representative of this company in October of last year, in an interview in Paris, commented to me far more on the value of hydraulic limes for sea water purposes than on any of the particular qualities of the alumina cement.

At the present time this cement is being manufactured alone by the LaFarge Company. No manufacturers in any other place in the world are at the present time producing it commercially. M. Jules Bied, who was formerly the Technical Director of the La Farge Company, undoubtedly developed this cement abroad, independently of any work in this country. He left the LaFarge Company and organized another company, which was engaged in its manufacture. However, at the time I visited him last October his company was not engaged in manufacturing, due to shortage of power, but were contemplating resuming manufacturing at another point on the completion of a plant. What effect his death will have upon this is not known. The LaFarge Company was manufacturing at the rate of 30,000 tons (180,000 barrels) per annum, but expected to have equipment in place so that by the first of the present calendar year they would be producing at the rate of 50,000 tons per year.

It should be remembered that these cements are manufactured from bauxite and limestone. In other words, instead of using clay and limestone as would be done in portland cement, a material much higher in alumina than clay must be used. This has been found in the bauxites too poor to be used in the manufacture of metallic aluminum. The source of this raw material is far more limited than clay. This is true in France as well as in this country, and as a consequence the price at the present time in France is about two and one-half times that of portland cement. This matter of the distribution of bauxite in this country must not be lost sight of. The nearest workable source to the New York district would be Georgia, and to Chicago would be Arkansas. The distances of these from the markets can be appreciated. The same condition exists in France. The bauxites are available in only the southernmost parts of France, but France is a relatively small country and the transportation charges for the haul from the source to the market would be considerably less than in this country.

You are all possibly aware that the Bureau of Standards published in 1919, at the meeting of the American Society for Testing Materials, some data it had obtained in testing these cements, made in its own experimental cement plant. This data was supplemented in 1921 by further data showing, as is well known, the rapid hardening but not rapid setting of these cements, and the high strengths that they consequently attain at early periods. The data thus obtained further showed that there was but a slight gain in strength with age, and that in the presence of moisture there was a falling off in strength at later periods. This has never been denied abroad, and M. Bied in a publication of his devotes about two pages to a discussion of the Bureau's work, in which he not only does not deny this quality of the alumina cement, but also calls attention to the fact that the French have found the same result, but at the end of their later periods of test the alumina cement is still higher testing than portland cements. To this we agree. At the present time the Bureau is making another series of burns of this cement in its own plant, in which it hopes to study further the reaction of these cements towards sea water, sulphate salt solutions, and in reinforced-concrete.

It is not to be inferred that we are pessimistic in regard to the qualities of this cement. On the contrary, we are very enthusiastic in regard to these, but it were well to more thoroughly investigate and consider their qualities before becoming over-enthusiastic in all respects. There are several unquestionable fields open to their use. Their application, as abroad, in city street repair work and city street paving would insure quite a demand and would justify their extra cost. Many repair jobs are of the same general nature. Where quick construction is demanded they would again find a large use. If they should be shown to be resistant to

sulphate water another very large field would be laid open. Though it has not yet been proved conclusively that they can be applied in this latter field, yet should it be proven they will have filled a want that would justify all that is being hoped of them.

It is rather strange that the interest in these cements should be aroused in this country through what has been done in France. Spackman and Lazell took out the patents in this country covering the manufacture of these in 1908 and 1912, yet nothing was done towards utilizing these, until the latter part of this last year in this country. Publications were issued in this country and abroad as early as 1919 showing very conclusively certain of their properties, yet it was the impetus of foreign usage which alone has aroused any interest in this country.

FIELD TESTS OF CONCRETE.

By John G. Ahlers* and Stanton Walker.**

Tests made during the summer and fall of 1923 in New York City through the co-operation of the New York Group of Contractors and the Structural Materials Research Laboratory, Lewis Institute, Chicago.

Since the organization of the American Concrete Institute, twenty years ago, the uses of concrete have developed to such an extent that from being considered merely as a substitute for stone masonry in the construction of massive work, it has become one of our most important structural materials. The annual production of 22,300,000 bbl. of cement in the United States in 1903, and of 137,000,000 bbl. in 1923 gives some idea of the enormous increase in the volume of concrete manufactured.

Researches into the properties of concrete have been under way for many years by several investigators in the United States and Europe; but only within the past few years has extensive and accurate information, applicable to the production of concrete in the field, been obtained. In the summer and fall of 1923 strength tests of concrete were made during the period of construction of five reinforced-concrete buildings in New York City. These tests were an outgrowth of an extensive series of field investigations undertaken to establish means of introducing into practice some of the improvements in the technique of concrete-making, which recent researches have indicated are desirable and to determine the practicability of securing the compressive strength desired in concrete produced under field conditions.

The first of these field investigations was planned and carried out by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, in co-operation with the General Committee of Contractors. The General Committee of Contractors was formed in 1921 under the joint chairmanship of M. J. Whitson, of Stone & Webster, Inc., and M. M. Upson, of the Raymond Concrete Pile Co. In May, 1922, the New York Group of Contractors was appointed as a Sub-Committee of the General Committee under the chairmanship of John G. Ahlers, to carry out similar field investigations in the New York territory, independently of the General Committee.

---

*Chairman New York Group of Contractors; Secy.-Treas. Barney-Ahlers Construction Corporation, New York City.

**Associate Engineer, Structural Materials Research Laboratory, Chicago.

Three major investigations have been carried out: two under the auspices of the Joint Committee and the General Committee, and one through the co-operation of the New York Group of Contractors and the Structural Materials Research Laboratory, as follows:

COOPERATIVE FIELD TESTS OF CONCRETE.

Investigation.	Date (1923).	Cooperating Parties.	Owner of Building.	Contractor.
I	April to July	General Committee of Contractors Joint Committee Bureau of Standards Structural Materials Research Lab.	Victor Talking Machine Co., Camden, N. J.	Stone & Webster, Inc.
II	Aug. to Oct.	New York Group of Contractors Structural Materials Research Lab.	New York Telephone Co. New York Giants Ward Baking Co. R. H. Macy Co. New York Telephone Co.	Turner Const. Co. Post & McCord, Inc. White Const. Co. Barney-Ahlers Const. Corp. Foundation Co.
III	Oct. to Dec.	General Committee of Contractors Joint Committee Bureau of Standards Structural Materials Research Lab.	Central Railroad of New Jersey, Jersey City,	Henry Steers, Inc.

This report covers only the tests listed under Investigation II in the above tabulation. The data of Investigations I and III will be reported through other channels. The members of the Committee of the New York Group of Contractors are: John G. Ahlers, chairman, Barney-Ahlers Construction Corp.; W. D. Binger, Thompson & Binger; J. E. Torrey, John W. Ferguson Co.; Aubrey Weymouth, Post & McCord, Inc.; W. Vaile, Industrial Engineering Co.; F. E. Rogers, Fred T. Ley & Co.; and J. B. Wright, American Concrete Steel Co. In carrying out these tests the contractors were represented by John G. Ahlers; the Structural Materials Research Laboratory by D. A. Abrams, Professor in Charge, and Stanton Walker, Associate Engineer. The tests were supervised by Stanton Walker. George Conahey and F. J. Rice were assistant testing engineers.

The expense of the investigation was borne by the New York Contractors and the Structural Materials Research Laboratory.

Valuable assistance was rendered by Columbia University in making available complete facilities for curing and testing the 6 by 12 in. concrete cylinders.

Acknowledgments are due the following sand and gravel companies for contributions to the Contractors' fund for carrying out these tests:

Kittanning Sales Co.
Nassau Sand and Gravel Co.
Rosoff Sand and Gravel Co.
Henry Steers, Inc.

OUTLINE OF TESTS.

A tentative program of tests was prepared after a series of conferences and was approved by the New York Group of Contractors.

It provided for carrying out tests on five different jobs, on one of which the major portion of the specimens were to be made. It required that eight 6 x 12-in. concrete cylinders for compression tests at 7d., 28d., 3m., and 1 y. be made at intervals of approximately 45 minutes for at least 20 working hours of concreting on the major job, and at least 10 hours of concreting on the four auxiliary jobs.

Concrete samples were to be selected from batches made under the usual operating conditions, and in accordance with ordinary methods of preparing the mix. After a study of the conditions on the job, samples were to be taken from batches of concrete prepared in accordance with scientific methods of designing the mix, in an endeavor to improve the uniformity and quality of the resulting concrete. The same materials were to be used in both cases.

The tests were to be carried out with practically no interference with job operations. Provisions were made to obtain complete information on the quality and uniformity of cement and aggregate, workability of concrete as measured by the slump and flow tests, time of mixing of concrete, and proportions of cement, water and aggregate.

Due to special conditions, it was impossible to adhere rigidly to the above program.

DESCRIPTION OF TESTS AND JOBS.

This investigation involved the manufacture of about 650 6 x 12-in. concrete cylinders for compression tests, sieve analyses of about 190 samples of aggregate, and the numerous miscellaneous determinations necessary to furnish complete information from which an intelligent analysis of the test results could be made.

The test data are given in Tables 1 to 18 and Fig. 9 to 11. All of the tests were made on reinforced-concrete structures for which concrete of a relatively high degree of workability was required. In general, the slump was 6 to 7 inches. For four of the jobs the aggregate was siliceous sand and pebbles from Long Island; for the fifth (Job E below) the aggregate was sand and pebbles of a dolomitic nature from Marlborough-on-the-Hudson River. Portland cement was used. Most of the concrete tested was of mixtures assumed to be equivalent to the usual 1 : 2 : 4. All concrete was machine-mixed in batch mixers. Table 1 gives a summary of miscellaneous information concerning each job.

A brief description of each job and an outline of the tests carried out, together with references to the tables and diagrams in which the results are recorded, is given below.

JOB A.—The major portion of the tests were carried out on Job A, selected as headquarters. It is a four-story reinforced-concrete building at Rockaway Avenue and Riverdale Street, Brooklyn, built by the Turner Con-

struction Co. for the New York Telephone Co. Fig. 1 is a photograph of the building after the concreting was finished. The work required about 7000 cu. yd. of concrete. The aggregate was "Ready-mix" gravel from Port Jefferson, L. I., furnished by the Kittanning Sales Co. Tests were made from September 12 to October 16, 1923.

For the slabs and certain of the columns the average mix used was 1 volume of cement to about 6½ volumes of aggregate, measured loose in a damp condition. This is equivalent to about a 1 : 7 mix by dry weight and a 1 : 5½ mix by dry and puddled volume. For certain columns a richer mix of about 1 : 5⅓ damp and loose was used. Each batch for the 1 : 6½ mix consisted of 5 sacks of cement and about 3500 lb. of mixed aggregate. For the 1 : 5⅓ mix 6 sacks of cement and about 3500 lb. of aggregate were used.

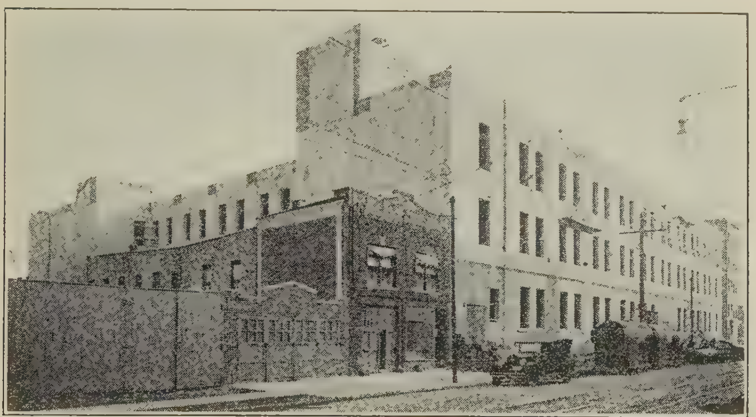


FIG. 1.—"JOB A" AFTER CONCRETING WAS FINISHED.
New York Telephone Co. Building, Rockaway Avenue and Riverdale St., Brooklyn, built by Turner Construction Co.

Forty-two batches of concrete were sampled and 291 6 x 12-in. concrete cylinders made for compression tests. For about half of the batches 8 cylinders were made from each sample for test at the four ages. For the other batches, in general, only two cylinders were made for test at 28 days. The workability of the concrete was measured for each batch by the slump and flow test. The time of mixing was determined for 16 of the test batches, and the quantity of mixing water recorded for 18 batches. Sieve analyses were made on 53 samples of aggregate selected from the stock pile and at the mixer. The results of the compression tests of the cylinders and miscellaneous data of the concrete batches are given in Table 3. The sieve analyses and miscellaneous tests of the aggregate are given in Table 9. A few tests were made to find the effect of sampling the concrete at the mixer, from the chutes, and from the slab forms. See Table 8.

Aggregate was delivered in scows to within about one mile of the job. It was unloaded by a clam-shell bucket to overhead bins from which it was loaded into trucks and hauled to the job. From the trucks it was dumped into the basement and then raised by means of a belt conveyor to a stock pile retained by vertical bulkheads. The aggregate for each batch was discharged through a gate near the bottom of the bulkhead into the metal measuring hopper on the mixer.

The concrete was mixed by a 1 cu. yd. barrel-type mixer, and hoisted and distributed by a tower and chuting system. Portland cement in sacks was handled from a storage pile to the mixer by a roller conveyor. The mixing water was measured in a 50-gal. barrel equipped with a calibrated floating gauge.

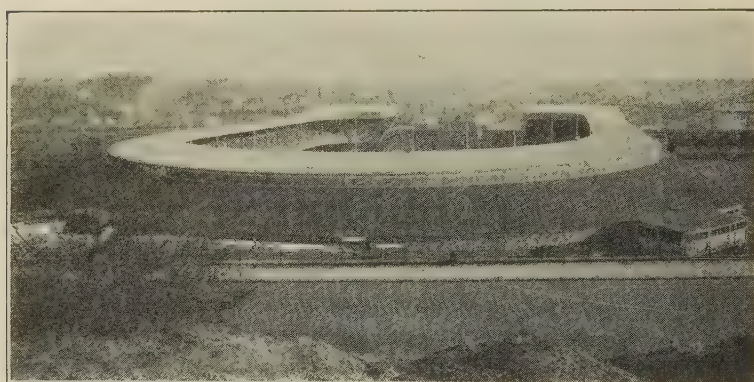


FIG. 2.—POLO GROUNDS STADIUM—JOB B.

Addition to East End, built by Post & McCord, Inc.

Although wide variations in grading of aggregate occurred, it was not practicable, because of the frequency and irregularity of these variations, to effect an economy by corrections to the grading of the "Readymix" aggregate. It appeared from the results of the 7-day tests that the proportions in use would, on the whole, produce the strengths required and therefore no changes were recommended.

JOB B.—Job B, under construction by Post & McCord, Inc., was at the Polo Grounds in New York City, and consisted of an addition to the stadium, which required about 7000 cu. yd. of concrete, and increased the seating capacity from about 35,000 to 55,000. Tests were started on August 6, when only about 300 cu. yd. remained to be placed, and continued until August 14. Fig. 2 is a view of the completed structure.

Sand and pebbles from the vicinity of Port Washington, L. I., furnished by the Lenox Sand and Gravel Co., were used as aggregates. The mix

specified by the architect was 1 : 1¾ : 4. Calibration of the measuring devices showed the average mix to be:

By damp and loose volume	1 : 1.8 : 3.8
By dry weight	1 : 1.5 : 4.3
By dry and puddled volume	1 : 1.3 : 3.7

For details of results of calibration of measuring devices see Table 12.

The maximum size of the aggregate was about ¾ in. A study of the sieve analyses showed these proportions to be economical for the conditions on this job and, therefore, no changes were recommended.

Samples of concrete were taken from 14 batches. One hundred and five 6 x 12-in. cylinders were made for compression tests at 7d., 28d., 3m., and

FIG. 3.—LAYOUT OF CONCRETE PLANT—JOB C.

Ward Baking Co. Building, 142d and Wales St., New York. One-story addition built by the White Construction Co.

ly. The workability of the concrete was measured for each batch by the slump test. The flow test was not made. Sieve analyses were made of 13 samples of sand and 13 of pebbles. The results of the compression tests of the concrete cylinders and miscellaneous data of the concrete batches are given in Table 4. The sieve analyses and moisture contents of the aggregates are given in Table 10 and the unit weights in Table 11.

The aggregates were hauled to the job in trucks, dumped on the pavement, and then transferred to the loading skip of the mixer by wheelbarrows. The concrete was mixed in a 10 cu. ft. barrel-type mixer, from which it was hoisted by means of a tower to an overhead hopper. From

the hopper the concrete was delivered to the forms by chutes or buggies, depending on the location of the work to be concreted.

The water for each batch was measured in a barrel equipped with a floating gauge. A constant quantity of water, gauged by the float, was put into each batch and then small quantities were added to obtain the proper consistency as judged by the mixer operator.

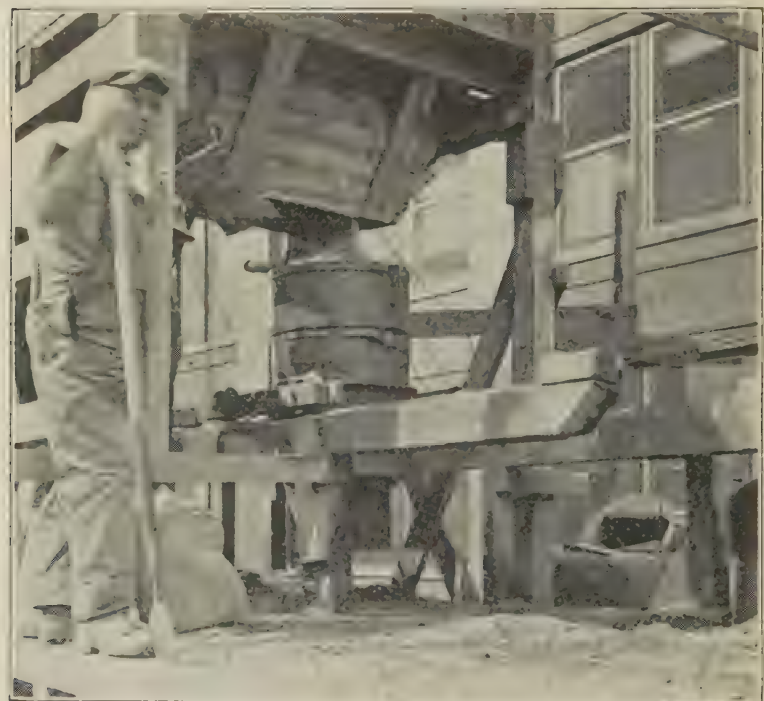


FIG. 4.—DEVICE USED ON JOB C FOR MEASURING SAND INUNDATED.

JOB C.—Job C was a one-story addition to the Ward Baking Co. building at 142d and Wales Streets, New York City, built by the White Construction Co. The aggregates were sand and pebbles from the vicinity of Port Washington, L. I., furnished by the Lenox Sand and Gravel Co. Preliminary studies of the aggregates were made before the concrete work was started, and the economical proportions to produce a slump of about 6 in. using a water-ratio of 1.0 were computed and checked by small trial batches. For the first day of concreting a mix of 1 : 2.1 : 4.4 by weight was used. Due to a change in the grading of the aggregate it was thereafter altered to 1 : 2.3 : 4.7. Tests were started on August 24 and continued

until September 7. One hundred and thirty-six 6 x 12-in. cylinders were made from 23 batches of concrete for test at 4 ages. The workability of the concrete was measured by both the slump and flow test.

The time of mixing was recorded for most of the batches. A fixed quantity of mixing water was used throughout (water-ratio 1.0). This was made possible by the special inundation measuring device for sand, described below. Sieve analyses were made of 22 samples of sand, and 21 samples of pebbles. The data of the tests are given in Tables 5, 13, and 14.

FIG. 5.—METHOD OF CHARGING MIXER WITH INUNDATED SAND—JOB C.

The aggregates were hauled in trucks and dumped on the pavement. The pebbles were transferred from the stock pile to the mixer in wheelbarrows. A bucket elevator lifted the sand to an overhead bin, from which it was discharged to the device designed by R. L. Bertin, Chief Engineer of the White Construction Co., for measuring sand under water. This apparatus consisted of a metal can of approximately 6 cu. ft. capacity divided into upper and lower compartments. The bottom of the lower compartment was adjustable so that the volume could be changed. The volume of the upper compartment could be changed by means of a collar

at the top. The operation consisted of admitting water into the can until the lower compartment was filled and sufficient water was contained in the upper compartment to completely inundate the quantity of sand required for a batch of concrete. The partition between the upper and lower compartments permitted the passage of water but not of sand. The water of inundation together with that contained in the lower compartment made up the mixing water. The can was mounted on a track in such a position that it could be moved to the mixer, tilted and its contents dumped into the mixer. Fig. 3, 4, and 5 are views of the concrete plant and of the sand measuring device. Complete description of the principles involved in

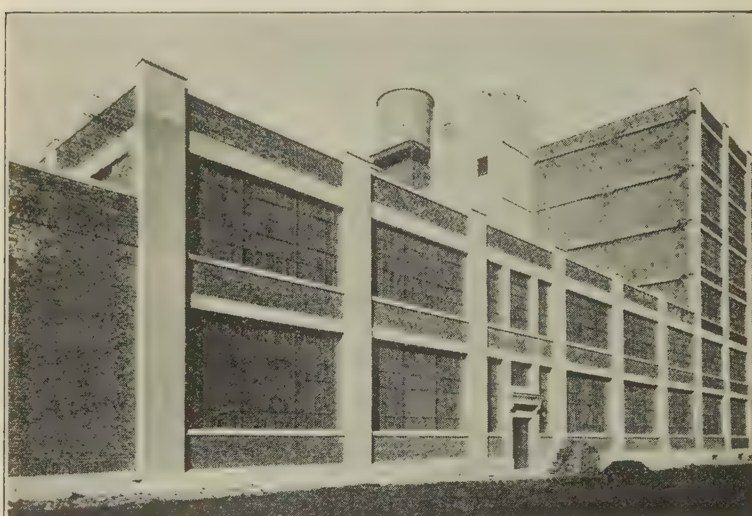


FIG. 6.—JOB D AFTER COMPLETION.

R. H. Macy Co. Warehouse, Long Island City, built by Barney-Ahlers Construction Corp.

this method of measurement of sand is given in a paper by R. L. Bertin, "Method for Measuring Sand in Inundated Condition," p. 404, 1922, Proceedings of the American Society for Testing Materials, and in "Inundation Methods for Measurement of Sand in Making Concrete," by G. A. Smith and W. A. Slater, in the 1923 Proceedings of the American Concrete Institute, page 222. The concrete was mixed in a ½ cu. yd. barrel-type mixer, and discharged directly into buggies which were carried to the work four stories above by means of an outside elevator.

JOB D.—Job D was a warehouse built at Long Island City for R. H. Macy & Co. by the Barney-Ahlers Construction Corp. Fig. 6 is a view of the building after completion. The aggregate was "Ready-mix" gravel from the same source as that used on Job A.

The average mix for most of the structural concrete (except for certain of the columns and the finish course of floor slabs) was approximately 1 : 6 damp and loose. Although wide variations in grading occurred, it was not practicable, because of the frequency and irregularity of these variations, to effect an economy by corrections to the grading of the "Ready-mix" aggregate. It appeared that the proportions in use would, on the whole, produce approximately the strengths required and, therefore, no changes were recommended.

Two batches of concrete were sampled on each of 4 different days, from September 28 to October 4. Eight cylinders were made from each sample for test at ages of 7 days, 28 days, 3 months, and 1 year. The time of mixing and quantity of mixing water was recorded for each batch. The workability of the concrete was measured by the slump test. The flow test was not made. Sieve analyses were made on 13 samples of aggregate selected from the stock pile and at the mixer. The data of the tests are given in Tables 6 and 15.

The aggregate was hauled to the job in trucks, dumped on the ground and then transferred by a clam-shell bucket to a stock pile and to an overhead bin. Fig. 7 is a photograph of the stock pile and bin. The aggregate for each batch was discharged from the overhead bin into a metal hopper attached to the mixer. A separate bin was provided for sand for use in the finish course on the floor slabs. It was available for addition to the "Ready-mix," but was not used for this purpose during the time covered by these tests.

The mixer used was a barrel-type of approximately 1 cu. yd. capacity. The concrete was hoisted in a tower and dumped into an overhead hopper, from which it was transported in buggies to the place of deposit. The water was measured in a 50-gal. barrel.

JOB E.—A few tests were made during the concreting of foundations for the New York Telephone Co. building, located between Barclay and Vesey Streets in down-town New York. The concrete work was done by the Foundation Co. "Ready-mix" gravel from the Rosoff Sand and Gravel Co., at Marlborough-on-the-Hudson, was used as aggregate.

The concrete contained about 6.25 bags of cement per cu. yd. This is equivalent to a mix of approximately 1 : 4 by dry and puddled volume. Samples were taken from the forms during the concreting of foundations. They were taken at 11 different times at intervals of not less than 30 minutes over a period of 5 days from September 14 to September 20.

Sixty 6 x 12-in. cylinders were made. The slump was measured and 4 or 12 cylinders were made for each sample of concrete and equally divided among the 4 test ages. Eighteen samples of aggregate for sieve analyses were selected from scows immediately before they were unloaded. The results of the compression tests and of the slump tests are given in Table 7. The sieve analyses are given in Table 16.

The aggregate was transported from the gravel plant on scows to within about one mile of the job. From the scows it was loaded by a clam-

shell bucket into overhead bins and from there hauled in trucks to the job, where it was dumped into a hopper leading to a bucket elevator. The elevator carried the aggregate to overhead bins, from which it was discharged into the hopper of the mixer. The concrete was mixed in two barrel-type mixers, each of approximately one cu. yd. capacity. From the mixer it was discharged into bottom-dump buckets carried on industrial trains which were pulled by gasoline engines to the vicinity of the work to be concreted. The buckets were lifted to above the forms by a derrick and the concrete dumped. The mixing water was not measured.

Laboratory Tests of Cement.—Samples of cement were selected during the course of the tests on each job and shipped to the Structural Materials Research Laboratory for test. The results are given in Tables 17 and 18.

TEST METHODS.

The tests were made in accordance with the recommendations of the American Society for Testing Materials where they were applicable and the experience gained by the Engineer of Tests during his connection in a similar capacity with the investigation carried out during the construction of Victor Talking Machine Co. Building No. 10, at Camden, N. J. The Camden investigation had been completed only a few weeks prior to starting the tests described in this paper.

The operations involved in obtaining the necessary information for the design and control of the concrete proportions are:

Aggregate
- Sampling
- Sieve analyses
- Unit weight
- Moisture content
- Absorption
- Colorimetric test for organic impurities

Calibration of Measuring Devices for Aggregate, Water and Cement

Design of Proportions

Concrete
- Sampling
- Determination of workability of concrete by slump or flow test

Compression Specimens
- Molding
- Curing
- Testing

To secure results which are indicative of the quality of the aggregates and concrete, the tests must be carried out with careful attention to detail. The art of designing and testing concrete is one that requires considerable experience and an appreciation of the effect of variations in the proportions of the concrete on its strength and other characteristics.

A brief description of the methods employed for the New York tests is given below. These methods can be applied successfully to the general problem of field control of concrete.

Sampling Aggregates.—Great care must be exercised to secure samples of aggregate which are representative of materials to be used. This is particularly true in the case of "Ready-mix" and coarse aggregates, for these, when handled, tend to segregate. In general, samples of aggregate were selected at the mixer for each batch from which compression cylinders were made.

When the aggregates were measured in wheelbarrows, a sample weighing about 50 lb. was secured by putting an occasional shovelful into a pan as the barrows were being filled. This method was used for both the sand and pebbles on Job B, and for the pebbles on Job C.

When sand was discharged from an overhead bin into the measuring device, a sample weighing about 10 to 15 lb. was secured by holding a pan in the stream of sand. Where possible 30 to 50-lb. samples of "Ready-mix" and coarse aggregates were obtained by the same method. Frequently this was impossible due to the force of the aggregate stream. In such cases the sample was shoveled from about 6 in. below the surface of the aggregate in the measuring hopper. To obtain preliminary information before concreting was started, samples were selected from the stock pile. Each sample was made up of aggregate taken from several spots about midway between the top and bottom of the pile, and at a depth of 12 to 24 in. from the surface where segregation was least.

For Job E, the samples of aggregate were selected at the scow before it was unloaded. About three samples were taken from each scow, one near each end and one near the middle, at a depth of not less than 2 ft.

Sieve Analyses.—The sieve analyses were made on dried samples, using the sieves recommended by the American Society for Testing Materials. (See for example Table 9). The sieves were shaken by hand. The amounts retained on each sieve were determined by weight. For coarse aggregate the entire sample (about 25 to 50 lb.) was tested, and for sand, about 500 grams (approx. 1 lb.), which was selected by quartering. The fineness modulus was used as a measure of the grading of the aggregate. It is the sum of the percentages in the sieve analysis divided by 100.

Unit Weight of Aggregate.—The weight per cu. ft. was determined for two conditions:

(a) Aggregate damp and loose approximating condition in measuring hopper. The test was made by shoveling average samples of the aggregate into a 1 cu. ft. cylindrical measure in such a way as to obtain as closely as could be judged the degree of compactness in the measuring hopper.

(b) Aggregate dry and puddled in cylindrical measure in accordance with Standards of the American Society for Testing Materials (C 20-21). The determination was made by filling a cylindrical measure having height equal to diameter, in three layers, and puddling each layer 25 to 30 times with a ⅝ in. round rod pointed at the lower end.

Moisture Content in Aggregate.—The moisture content was determined by drying to constant weight the samples used for sieve analyses.

Absorption of Aggregate.—Knowledge of the quantity of water absorbed by the aggregate is necessary in computing the quantity of mixing water. This determination cannot be made accurately in the field, however, and for this series of tests the absorption was not determined, because from previous knowledge of the materials used it was known to be low. Where absorption determinations are required they should be made in a laboratory.*

Colorimetric Test.—The colorimetric test for organic impurities in sand was made by digesting a sample in a 3 per cent solution of sodium hydroxide (NaOH) for 24 hours, and observing the color of the liquid above the sand. For a more complete description of this test see "Abrams—Harder Field Test for Organic Impurities in Sands," Proc. Am. Soc. Testing Mat., 1919, Part I; also "Tentative Method of Test for Organic Impurities in Sands for Concrete," Proc. Am. Soc. Testing Mat., 1921.

Calibration of Measuring Devices.—The proportions actually in use were in general, determined by weighing the quantities of materials in average batches. In other cases, where facilities for doing this were not readily available, the quantities were determined by accurate measurements of volume. The water measuring devices were marked so that the amount of mixing water could be accurately determined.

Sampling Concrete.—In order that the concrete tests may be truly representative of the quality of concrete being placed, great care must be exercised in the selection of samples. The samples for these tests were obtained as near the point of deposit as practicable. For concrete distributed by chutes, the samples were taken by holding 3-gal. pails under the discharge end of the chute. Each sample consisted of a mixture of 5 such pails full of concrete from one batch.

Where buggies were used to distribute the concrete, the samples were obtained by means of specially designed galvanized-iron sampling boxes of about 1 cu. ft. capacity placed in the top of the buggy. The dimensions of the boxes were such that they filled simultaneously with the buggies. Each sample consisted of two such boxes of concrete from one batch. Fig. 8 is a photograph of these sampling boxes, taken on the Victor Talking Machine Co. job at Camden, N. J.

On Job E, where massive foundations were being erected, the samples were taken after the concrete had been deposited in the forms.

The Standard Method of the American Society for Testing Materials, for making field tests of concrete (C 31-21), requires that all samples be selected from the forms. For this work such a method was not followed because it was necessary to obtain the samples from definite batches, which is, in general, impractical if the concrete is first placed in the forms. In

*For methods of making absorption tests see Report of Committee C-9, Proceedings of American Society for Testing Materials, vol. xx, p. 301, 1920.

many cases the method of selecting samples from the forms is objectionable also, because of the difficulty of obtaining representative samples. In the case of columns and certain other reinforced members, it is difficult to obtain a sample of any kind.

Workability.—The workability of the concrete was determined by both the slump and flow tests.

The slump test was made using a truncated metal cone 4 in. in diameter at the top, 8 in. at the bottom and 12 in. high, in accordance with the Tentative Specification, D 62-20 T of the American Society for

FIG. 7.—AGGREGATE STOCK PILE AND STORAGE BIN—JOB D.

Testing Materials adopted by the Joint Committee as a standard. A revision of this tentative specification, D 138-22 T provides that the slump test shall be made 3 minutes after molding the specimen. The former method was used and seems preferable. Fig. 8 is a view of the slump test being made. For convenience the flow table was used as a working platform.

The flow test was made using a flow table loaned by the U. S. Bureau of Public Roads.* The test made in accordance with recommendations of the Bureau of Standards, consists of jigging a cone of concrete on a special table (Fig. 8) and measuring the increase in bottom diameter. The con-

*For details of this flow table see "Inundation Methods for Measurement of Sand in Making Concrete," by G. A. Smith and W. A. Slater, Proc. A. C. I. vol. xix (1923), p. 227.

crete was molded in a truncated metal cone having a top diameter of 6⅔ in., a bottom diameter of 10 in., and a height of 5 in. The form was withdrawn from the concrete immediately after molding and the table raised and dropped ⅛ in. by means of a cam, 15 times in about 10 seconds. The increase in the base diameter expressed as a percentage of the original diameter, is the flow. The method employed at the Structural Materials Research Laboratory differs from the above in that a ½ in. drop is used, and the flow is taken as the final base diameter expressed as a percentage of the original diameter.

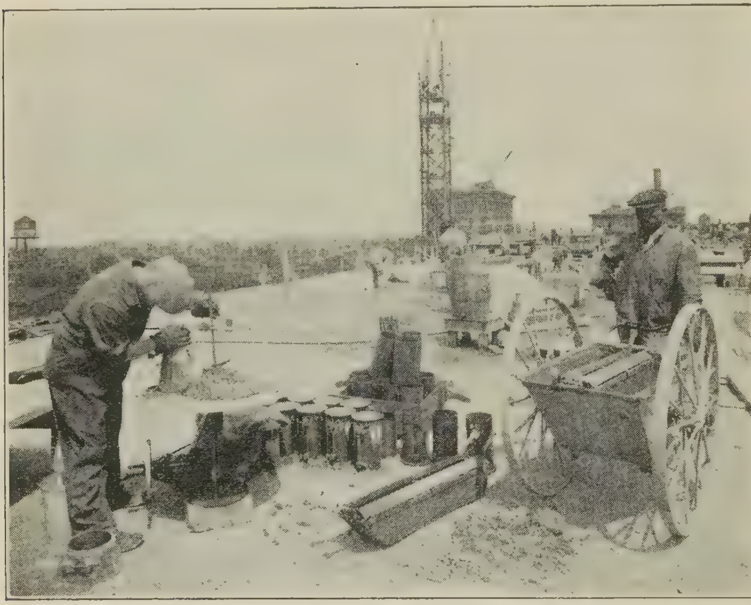


FIG. 8.—SLUMP TEST AND SAMPLING BOXES.

Molding Compression Cylinders.—The 6 x 12-in. compression cylinders were made on the work near the place where the concrete was sampled. The cylinders were molded in cylindrical metal forms set on machined cast-iron plates, in accordance with the recommendations of the American Society for Testing Materials. The specimens were removed from the forms after 16 to 20 hours and stored in damp sand until tested. The tops of the cylinders were capped with neat cement a few hours after molding, or with gypsum a few hours before testing, and a smooth surface formed by means of a machined plate.

Strength Tests.—The strength tests were made at the Testing Laboratory of Columbia University, using a 400,000 lb. Olsen or a 100,000 lb. Riehle testing machine. The cylinders were tested between steel plates. The load was applied through a spherical bearing block on top of the specimen.

DISCUSSION OF TESTS.

The data of the tests are given in Tables 1 to 18, and Fig. 9 to 11.

Strength of Concrete.—The concrete for the 5 jobs (except for certain of the columns on Job A) was of proportions assumed to be equivalent to the usual 1: 2: 4 mix. Table 2 summarizes data from Tables 3 to 7, and gives the average strengths for concrete of this mix for the 4 ages. At 28 days the average strengths ranged from 2120 for Job E to 2390 lb. per sq. in. for Job B. At 7 days the range was from 1330 to 1550 for the same jobs; and at 3 months from 2680 to 3300 for Jobs D and A respectively. Without exception the average strengths at 28 days were considerably in excess of the expected strengths arrived at by a study of the Joint Committee Table of Proportions. These expected strengths, computed from the same basis as the Joint Committee Tables are given in Table 2 as "minimum expected strength." Only 3 of the 179 cylinders tested at 28 days gave strengths lower than the "minimum" expected values. Two cylinders from one batch on Job D gave strengths 15 and 20 per cent lower than this value. For Job E only one cylinder was below the "minimum" expected value.

The "average" expected strength given in Table 1 is based on the water-ratio-strength relation for Fig. 1 in Bulletin 1 of the Structural Materials Research Laboratory. Jobs A, B, C and D gave average 28-day strengths as much as 30 per cent higher than the "average expected strengths." Job E gave an average 28-day strength about 12 per cent less than predicted from the average curve.

The strength of the concrete at 7 d. and 3 m. is discussed below:

Uniformity of Strength Tests.—The individual cylinders from a single batch gave consistent strengths at each age and, therefore, considerable confidence may be placed in the compression tests as indicating the quality of concrete in the structure. The mean variations in Table 2 and the curves in Fig. 9 give two different measures of the uniformity of the results for different batches. The table gives the percentages of mean variation, and the percentages of number of cylinders within the limits of 20 per cent of the average for the 7-d., 28-d. and 3-m. tests for each of the 5 jobs. Fig. 9 shows the cylinder strengths for each job platted in order of magnitude. The abscissas are percentages of total number of cylinders having strengths lower than the corresponding strength. The ordinates are the compressive strengths of the individual cylinders. Curves platted in this way are useful in showing clearly the maximum range in values and the number of tests falling within different limits.

The average of each of these two measures (the mean variation and the percentage of number of specimens falling within 20 per cent of the average) for the three ages at test arrange the different jobs in the same order according to uniformity of the results. The best uniformity measured by these methods was found on Jobs B and E; the mean variations were 8.5 and 8.1 per cent respectively. The greatest variations were found in the case of Jobs A and D; the mean variations were about 17.3 and 15.5

per cent. The variations for Job C were about midway between the two extremes (mean variation 11.8 per cent).

An analysis of the conditions on each of these jobs shows the causes for the differences in uniformity.

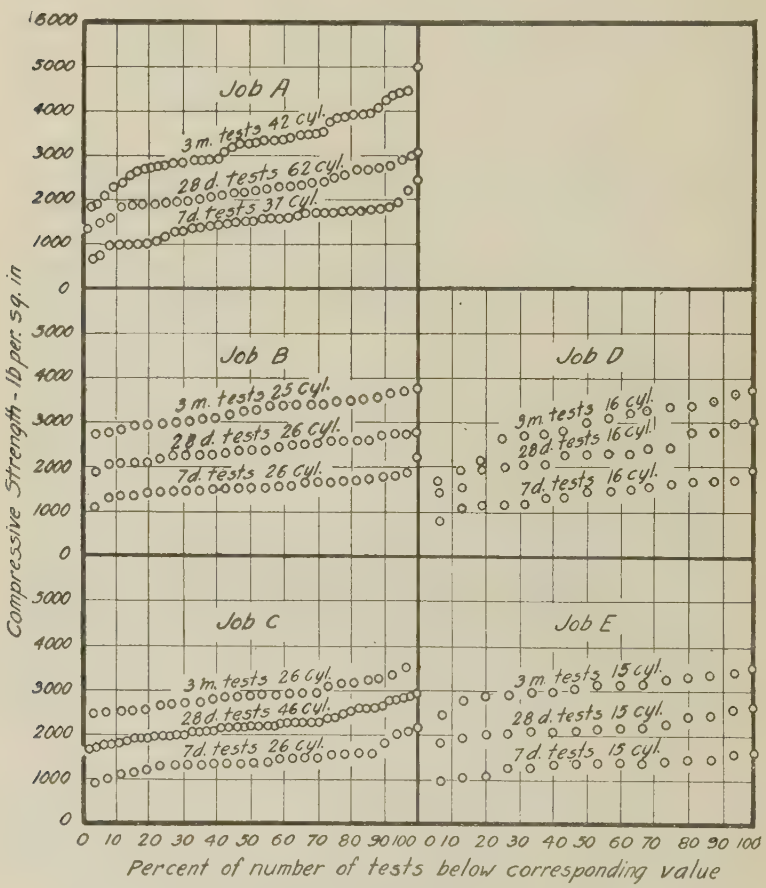


FIG. 9.—VARIATION IN STRENGTH OF CONCRETE.

Compression tests of 6 by 12-in. concrete cylinders arranged in order of magnitude.

A study of the "Ready-mix" aggregates used on Jobs A and D carried out prior to starting the tests, made it apparent that a non-uniform concrete was to be expected, due to the wide variations in grading. An inspection of the source of supply and method of preparation of this material, compared with the "Ready-mix" from other plants in the New York

District, showed that a more uniformly graded mixture could be produced than was furnished to these jobs. The authors understand that as a result of this investigation a plant which compares favorably with other "Ready-mix" plants in this territory, is being installed. Wide variations in grad-

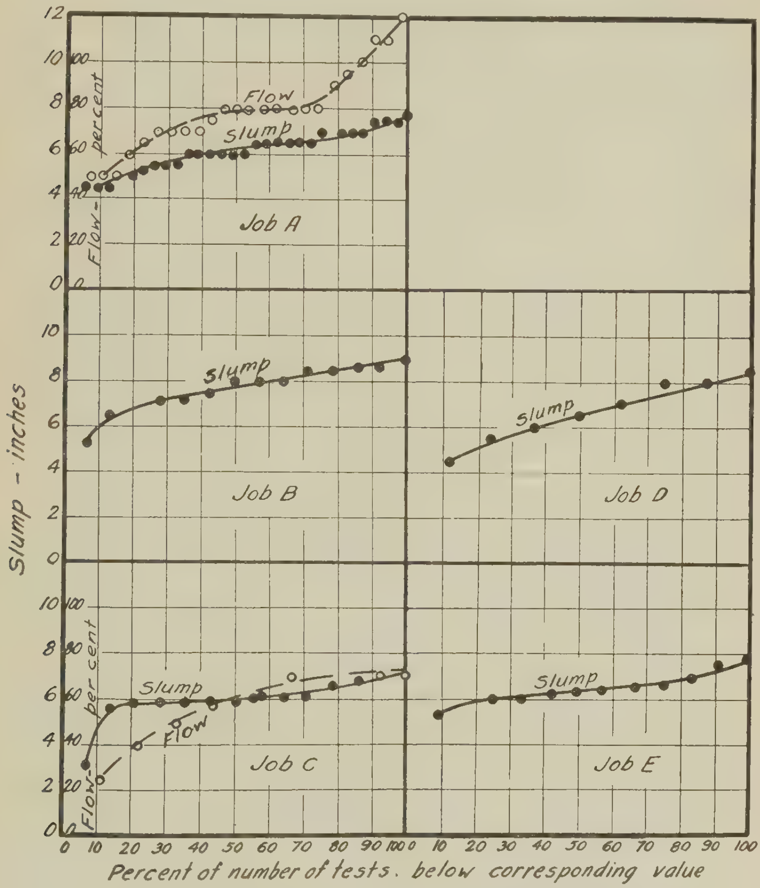


FIG. 10.—VARIATION IN SLUMP AND FLOW OF CONCRETE.
Slump and flow of concrete arranged in order of magnitude.

ing of this material were encountered for successive concrete batches on Jobs A and D. See Table 9 and Fig. 11. As a result, it was necessary to vary the quantity of mixing water to maintain a uniform workability. The method of measuring the materials permitted considerable variations in the quantities and wide variations occurred in the grading of the aggregate even from one part of the batch to another. However, the average

strengths obtained were satisfactory and in spite of the wide variations shown above only two of the 28-day cylinders for these jobs, gave strengths lower than the "minimum" value calculated on the same basis as the Tables of Proportions in the Progress Report of the Joint Committee. (See also

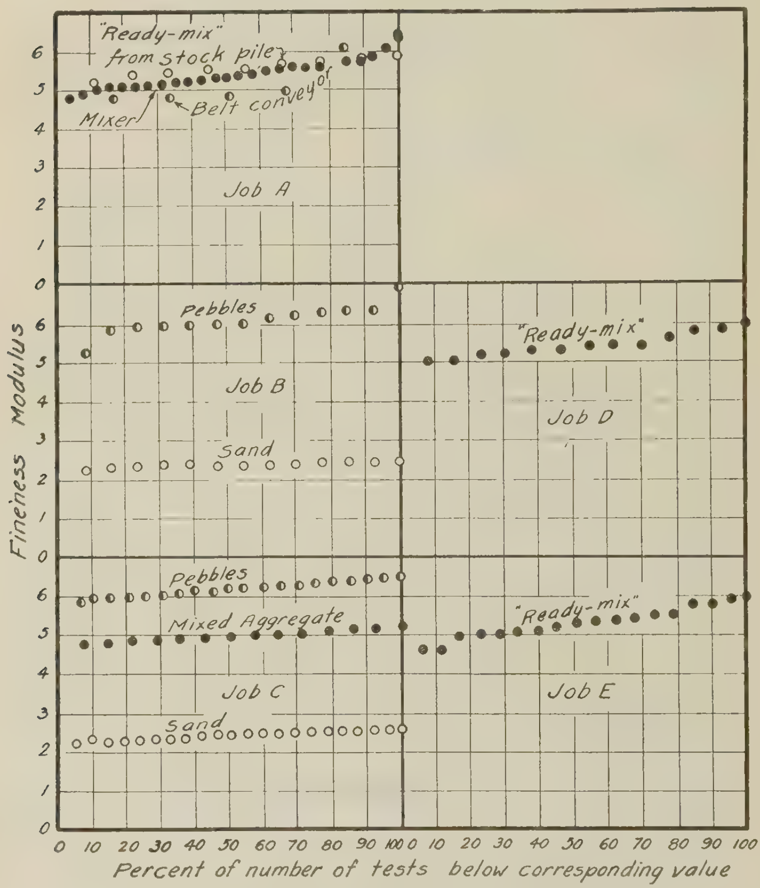


FIG. 11.—VARIATION IN FINENESS MODULUS OF AGGREGATE.

Fineness modulus of aggregate arranged in order of magnitude.

Bulletin 9 of the Structural Materials Research Laboratory.) The lack of uniformity for the strengths was due as much to the abnormally high strengths obtained as to those in the lower range.

The reasons for the uniform results on Job B seem apparent. The grading of the aggregates was fairly uniform. See Table 10 and Fig. 11. A fair degree of uniformity was maintained for the plasticity of the con-

crete. The variations in slump are not unexpected when the inherent inaccuracies of the test are considered. See Fig. 10. The aggregates were measured in carefully calibrated wheelbarrows and loaded under the direction of an architect's inspector whose sole duty was to supervise the mixing and proportioning. The concreting was done in comparatively small sections which did not involve rush work. Although no definite relation is shown between time of mixing and strength for these tests, it is noteworthy that none of the batches tested was mixed for less than 53 seconds, after all of the materials were in the hopper. Uniform weather also, undoubtedly, tended to eliminate variations in strength.

The causes for the greater uniformity in strength obtained for Job E are not as apparent as in the case of Job B. Information was not obtained in the same detail for this job as for the others. The fact that the concrete was selected from the forms and from mass concrete probably accounts to some extent for the small variations. Although the fineness modulus varied from 4.6 to 5.9 for the small samples used for sieve analyses, it is probable that much less variation in grading existed in the aggregates as used at the mixer. The material *appeared* reasonably uniform as it was charged into the mixer. It is unfortunate that time was not available to carry out the tests on this job in greater detail. While the mixing water was not measured, the samples of concrete taken from the forms for test gave slumps of good uniformity. See Fig. 10.

The less uniform strengths for Job C are due to the abnormally high strengths of batches 1 and 6. If these batches are omitted, 92 per cent of the 7-day tests, 90 per cent of the 28-day tests, and 97 per cent of the 3-months tests are within 20 per cent of the average. The results for this job would then compare favorably with the most uniform results obtained in this investigation. (Jobs B and E.)

Effect of Age on the Strength of Concrete.—Increases in strength with age were found for all tests in this investigation up to 3 months. A summary of the strength tests for each job based on Tables 3 to 7 is given in Table 2. The arithmetic average of the strengths for each job were 1420 lb. per sq. in. at 7d., 2240 at 28d., and 3020 at 3 m.

The following tabulation gives the average, maximum, and minimum ratios of 7-d. and 3-m. tests to those at 28 d. for each job and the mean variations of these ratios from the average.

The 7-day tests are most useful for the field control of concrete as the results are available while the conditions prevailing at the time the specimens were made are fresh in the minds of the persons interested. Approximately the same average ratios were obtained for each job. The mean variations and the percentages of number of values falling within 20 per cent of the average for the ratios of individual cylinders from the same batch, showed a good uniformity as compared with the strength tests at each age. It should be noted that these ratios will vary considerably with the conditions of test. Tests made at the Structural Materials Research Laboratory indicate that the ratio of 7-d. strengths to 28-d. strengths is

Job	Per Cent of Strength at 28 Days.			Per Cent of Values within 20 Per Cent of Average.	Mean Variation from Average Per Cent.
	Average.	Minimum.	Maximum.		
	7-DAY TESTS.				
A*	66	36	88	81	14.1
A†	66	48	81	85	12.1
B	64	47	85	85	9.2
C	63	44	77	92	10.1
D	62	50	81	94	11.0
E	60	46	72	87	12.5
Average	64	46	81	87	11.5
	3-MONTH TESTS.				
A*	137	102	169	75	12.8
A†	131	118	152	100	8.4
B	136	117	168	96	8.0
C	129	108	169	92	9.7
D	128	99	161	75	11.0
E	142	112	166	93	8.1
Average	134	109	164	88	9.7

* Mix 1–6½ damp and loose.
† Mix 1–5⅔ damp and loose.

generally nearer 50 per cent than the 64 per cent found for these tests. All of the specimens were cured in damp sand until test, and tested damp.

Tests made in the Joint Committee investigation on the Victor Talking Machine Co. building at Camden, N. J., indicate that at 28 days the compressive strength of field-made cylinders cured in damp sand is about the same as that of cores cut from slabs or compression specimens sawed from columns which had been cured under the same conditions as the structure. the above-mentioned tests of cores from slabs and specimens from columns as well as data from other sources* indicate that substantial increases in strength occur after 28 days. Laboratory tests show conclusively that con-

*See for example:

Effect of Age and Condition of Storage on Strength, by H. F. Gonnerman. Proc. Am. Concrete Inst., 1918.

Effect of Age on the Strength of Concrete, by D. A. Abrams. Proc. Am. Soc. Testing Mat., Part 2, 1918.

Ten-Year Tests Showing Effect of Age and Curing Conditions on the Strength of Concrete, by M. C. Withey. Eng. and Contr., vol. 54, p. 519, Nov. 24, 1920.

Effect of Age on the Strength of Concrete, by D. A. Abrams. Concrete, July, 1921.

Relation Between Molded and Core Concrete Specimens, by H. S. Mattimore. Eng. News-Record, vol. 88, No. 2, p. 73, Jan. 12, 1922.

A comprehensive bibliography of the literature of tests of concrete at different ages is available at the Structural Materials Research Laboratory.

crete increases in strength with age so long as it is not dried out and moisture is available for the progressive hydration of the cement. It is a reasonable conclusion, therefore, that tests of field specimens at later ages, say 3 months, are a better indication of the ultimate strength of the structure than tests at 28 days. This statement should not be construed as a recommendation for lowering present standards of quality of concrete.

Tests of Concrete Sampled at Different Points.—Table 8 gives results of tests of concrete taken from 4 different batches for which the samples were selected at 3 different stages in the concreting operations: (1) at the discharge of the mixer, (2) at the discharge of the chute, and (3) from the slab forms immediately after deposit. Although no great differences were found in the results of these tests, it is apparent that a sufficient number of batches was not sampled to give conclusive information. Practically the same strengths were shown for the samples taken at the mixer and from the forms. The average strength for concrete from the chute is somewhat higher than that from the other two points. The slump and flow were measured only for samples from the chute and form. The samples from the chute gave a lower slump and lower flow which is consistent with the higher strengths, and indicates that the differences are due to differences in the quality of the sample. No explanation is offered for these results other than that they are probably accidental. One theory that has been advanced from time to time is that the concrete receives additional mixing during its travel through the chutes which materially increases its strength. This apparently is not the reason for the higher strengths found in this case, for if this were so, a corresponding increase in strength would have been found for the concrete taken from the forms. The mean variation of the samples from the chute was 5.4 per cent as compared with about 9 per cent for the other two sets of tests.

Measurement of Quantities of Materials.—One of the most obvious reasons for the lack of uniformity in the strength of field-made concrete is the wide variation in quantities of materials which occurs from batch to batch, especially in the case of the aggregate and the water. A common means of measuring aggregate is the metal hopper with which the mixer is equipped. These hoppers are usually pyramidal in shape and have an area at the top of such dimensions that 1 or 2 in. difference in the level of the aggregate makes a material difference in the volume. In order to maintain the same workability the quantities of mixing water must be varied to correspond with variations in the volume of the aggregate. The moisture content of the aggregate also exerts an important influence on the quantity of the aggregate and of the mixing water. A number of different investigators have shown that sand bulks as much as 25 to 30 per cent for quantities of moisture commonly encountered under job conditions. The bulking affects the proportions of the concrete batch by changing the volume of the materials. The differences in moisture content changes the volume of mixing water. Great improvements can be made in the uniformity of field concrete if these factors are intelligently controlled. A method for

accomplishing this result, described above, was used on Job C and seems to offer a most promising means of solving the difficulty.

A great improvement over present common methods, which involves no radical changes, is the measurement of materials in hoppers having approximately vertical sides and placed in such a position that the aggregate can be readily struck off. The variations in moisture content on the job are usually small for short periods of time, and therefore, the element of bulking can usually be taken into account by varying the volume of damp and loose aggregate with changes in moisture content. Neither the bulking factor nor the amount of moisture in aggregate are as important in coarse aggregate as in sand.

Uniform proportions can also be obtained by measuring the aggregate in carefully calibrated wheelbarrows or buggies and striking it off.

Control of Concrete Proportions in the Field.—The principles outlined in the publications of the Structural Materials Research Laboratory, particularly Bulletin 1, were followed in determining what were the most economical proportions and in deciding whether the proportions in use on each job might be expected to give the required strengths. The principles involved in a consideration of the problems of this investigation may be briefly outlined as follows:

1. The strength of concrete is fixed by the quantity of mixing water expressed as a ratio to the volume of cement, so long as the concrete is workable.

2. The variation of other factors in proportioning concrete affect the strength only because they change the quantity of mixing water required to produce concrete of the desired workability.

3. For concrete manufactured under average conditions the equation $S = \frac{14000}{7^x}$ may be expected to represent the strength; where $S =$ compressive strength at 28 days lb. per sq. in. and $x =$ water-ratio; ratio of volume of water to volume of cement (an exponent).

4. The minimum strength to be expected is given by the equation $S = \frac{14000}{9^x}$ which is of the same form as that in 3 above.

5. Fineness modulus is a "yard stick" by which the concrete-making properties of aggregate, so far as grading is concerned, may be measured. The fineness modulus is the sum of the percentages in the sieve analysis divided by 100 when sieves No. 100, 50, 30, 16, 8, 4, ⅜ in., ¾ in., and 1½ in., etc., are used. An important characteristic of these sieves is that the clear opening of each is double that of the next smaller sieve.

The fineness modulus of a mixture of fine and coarse aggregate is directly proportional to the fineness moduli of the separate materials and the proportions in which they are mixed. If it is desired to mix two aggregates in the proper proportions to produce a given fineness modulus,

the volume of fine aggregate expressed as per cent of the volume of fine and coarse measured separately, may be calculated from the equation,

$$p = 100 \frac{A - B}{A - C}.$$

Where p = ratio of volume of fine aggregate to volume of fine and coarse aggregate measured separately; A = fineness modulus of coarse aggregate; B = fineness modulus of mixed aggregate and C = fineness modulus of fine aggregate.

6. The maximum permissible values of fineness modulus, as given in Table 3, Bulletin 1, provide a proper basis for calculating the ratio of fine to coarse aggregate.

Changes in proportions were made during the course of the work on Job C only. It was pointed out above that for Job B the most economical ratio of fine to coarse aggregate for the materials available were in use when the tests were started. The grading of the aggregate furnished throughout the course of the tests was uniform. On Jobs A, D and E "Ready-mix" materials were used, and it was not practicable to make corrections in the grading. Changes on Job C were made because of a marked difference in the grading of the coarse aggregate furnished after the concreting was started.

To maintain a uniform quality of concrete, it is necessary to use aggregate of uniform grading, or to make proper changes in the proportions with changes in grading. This investigation showed conclusively that it is not practicable to compensate for accidental variations in grading of aggregate from batch to batch. However, when the aggregate varies in grading for different scow loads or other relatively large lots, and each lot is uniform within itself, it is practical to vary the proportions in accordance with changes in grading. This necessitates constant inspection on the job by an engineer capable of estimating such changes, and it is felt that it will be found more economical to require that the aggregates be furnished uniformly graded throughout the course of the work, and to make the inspection at the source of supply. The specifications for grading should be drawn to conform to the most economical proportions for the materials available. It is neither necessary nor desirable to apply the same specification for grading in all cases, as concrete of the same quality can be made from materials of a wide range in grading, as well as other characteristics.

Aggregate Sources in the New York District.—Inspections were made of four aggregate plants which furnish a large proportion of materials used in the New York District. In general, these plants wash the bank-run aggregates and divide them into four to seven different sizes which are recombined in the required proportions. Usually the aggregate is separated into the different sizes by washing through screens then loaded into bins, from which it is discharged to belt conveyors and transported to the scow. The proportion of different sizes is controlled by the width of open-

ing of the gate from the bin. The mixing is accomplished by discharging the materials from the various bins to the continuous belt from which in turn it is discharged to another belt and then to a hopper above a flexible loading spout which deposits the material on the scow in thin layers.

CONCLUSIONS.

In applying the results of this investigation to the general problem of field control of concrete, the conditions under which it was carried out should be kept in mind. The tests were made on work under construction by high-grade contractors, in a district where aggregates of excellent physical qualities are used. Trained organizations of long experience and adequate equipment for proportioning and mixing the concrete as measured by present standards, were used.

The following conclusions may be stated:

1. On these five jobs, where trained organizations of long experience were used, the proportions and methods employed, based on experience only, gave concrete having an average strength from 6 to 18 per cent greater than that assumed for purposes of design. It is noteworthy in this conneetion that many of the objectionable practices which account for unsatisfactory concrete, were not encountered on these tests.

2. With the concreting methods used on these jobs, it would be fair to expect a uniformity of strength such that not less than 75 per cent of the test specimens would fall within 20 per cent of the average strength, and not more than 5 per cent would fall below a minimum value, computed on the basis of the Joint Committee Tables of Proportions, (also published in Bulletin 9, of the Structural Materials Research Laboratory).

3. A study of the proportions and the materials, carried out in the light of knowledge now available, should enable the engineer to estimate within 20 per cent the average strength of the concrete when made in accordance with the general practices followed on these jobs.

4. Estimates of the concrete strengths made for the 5 jobs on which tests were carried out when compared with the strengths obtained, show that the water-ratio offers a practical basis for estimating the strength of concrete, if aggregates of suitable physical structure are used.

5. The fineness modulus was an accurate "yard-stick" for measuring the grading of aggregate for the purpose of determining the economical proportions of fine to coarse.

6. The maximum permissible values of fineness modulus given in Table 3, Bulletin 1, of the Structural Materials Research Laboratory may be satisfactorily applied to building construction. These values of fineness modulus produce concrete which works somewhat more harshly than that to which the mechanics are accustomed, and in certain cases a campaign of education may be desirable before the maximum values given by Table 3 are used. Reduction of these maximum values by 0.25 produces concrete which is easily worked into the forms in building construction.

7. Concrete designed under the conditions encountered in this investigation gave strengths well above the values computed on the same basis as the Joint Committee Tables of Proportions.

8. The variations in strength shown by the individual specimens for different batches were in general, the result of accidental variations in the proportions; the satisfactory agreement of different specimens from the same batch indicate that the variations were not due to the test methods employed.

9. A device which will insure a uniform quantity of aggregate from batch to batch will be of material assistance in securing concrete of uniform strength. Hoppers having vertical sides and provided with a mechanical strike-off are practicable and offer great advantages over present methods. Uniformity in the volume of sand can be maintained by the inundation method of measurement.

10. In order to secure concrete of uniform quality with the least cost for proper control, uniformly graded aggregate should be furnished throughout the course of the work. The specifications for grading should be drawn to conform to the most economical proportions for the aggregate available. It is neither necessary nor desirable to apply the same specification for grading in all cases, as concrete of the same quality can be made from aggregates of a wide range in grading and other characteristics.

11. It was not practicable to adjust the proportions on the job to compensate for variations in the grading of the aggregate which occurred from batch to batch. However, it was feasible to modify the proportions to compensate for variations in grading occurring in different scow loads or other lots of considerable size, where each lot was uniform within itself.

12. For large concrete jobs preliminary studies should be made of the aggregate at the source of supply for the purpose of determining economical proportions.

13. Scientific control can best be accomplished by using a testing organization outside that of the owner or contractor. Provision in the contract for an equitable division of the expense of scientific control, will greatly facilitate such control.

14. The experience obtained during this investigation shows conclusively the vital importance of making tests on the job to determine the strength of the concrete used. This is evident from a consideration of the definite economies which may be effected in proportioning the concrete and in designing the structure to take advantage of the known strength of the structural material employed. Definite strengths can be obtained only by: (1) rigid inspection and (2) tests of the concrete.

TABLE 1.—MISCELLANEOUS INFORMATION ON JOBS.

Field tests carried out on five different jobs in New York in 1923.
Machine-mixed portland cement concrete having slump of about 6 to 7 in. used on all jobs.
Aggregate from Long Island, except that for Job E, which was from Marlborough-on-the-Hudson.

Job.	Owner.	Contractor.	Kind of Aggregate.	Mix by Volume* (Dry and Puddled).	Period of Tests (1923).	Number of 6 x 12-in Cylinders.	Data in Tables.
A	New York Telephone Co.	Turner Const. Co.	"Ready-Mix"	1:5½	Sept. 12 to Oct. 16	291	2, 3, 8, 9
B	New York Giants (Polo Grounds)	Post & McCord, Inc.	Sand and Pebbles	1:1 8:3.8	Aug. 6 to Aug. 14	105	2, 4, 10, 11, 12
C	Ward Baking Co.	White Const. Co	Sand and Pebbles	1:1.9:4.0 1:2.1:4.2	Aug. 24 to Sept. 7	136	2, 5, 13, 14
D	R. H. Macy Co.	Barney-Ahlers Const. Corp.	"Ready-Mix"	1:5	Sept. 27 to Oct. 4	64	2, 6, 15
E	New York Telephone Co.	Foundation Co.	"Ready-Mix"	1:4	Sept. 14 to Sept. 28	60	2, 7, 16

* Cement assumed to weigh 94 lb. per cu. ft.

TABLE 2.—SUMMARY OF CONCRETE TESTS.

Compression tests of 6 x 12-in. concrete cylinders made in field.
Mixtures approximately equivalent to 1:2:4; the richer mixtures tested for Job A omitted from average.
Machine-mixed portland cement concrete.
Aggregate from Long Island, except that for Job E, which was from Marlborough-on the-Hudson.
Values for expected strengths based on water-ratio-strength relation. The "minimum" expected strength calculated from equation, $S=\frac{14000}{9^x}$; the "average" expected strength from equation $S=\frac{14000}{7^x}$; where S=compressive strength at 28 days in lb. per sq. in., and x=water-ratio (an exponent). The first equation was used as the basis of the Joint Committee Tables of proportions (also published in Bulletin 9 of the Structural Materials Research Laboratory; the second equation is from Bulletin 1 of the Structural Materials Research Laboratory).
The "mean variations" are the averages of the variations of the individual strengths from the average strength of all cylinders for a given job tested at a given age, expressed as a percentage of the average strength.

Job.	Workability.		Water-Ratio.	Compressive Strength, lb. per sq. in.				Expected Compressive Strength at 28 days, lb. per sq. in.	
	Slump.	Flow, per cent.		7 day.	28 day.	3 mo.	Average.	Minimum.	Average.
A	6 1	80	1.1	1450 (37)[a] (62)[b] 19.7[c]	2130 (76)[a] (72)[b] 14.4[c]	3040 (34)[a] (85)[b] 17.8[c]	.... 73[b] 17.3[c]	1250	1650
B	7.7	..	1.0	1550 (26)[a] (92)[b] 10.9[c]	2390 (26)[a] (100)[b] 8.2[c]	3230 (26)[a] (100)[b] 7.4[c]	.... 97[b] 8.5[c]	1550	2000
C	5.9	60	1.0	1430 (26)[a] (70)[b] 14.7[c]	2180 (46)[a] (83)[b] 11.7[c]	2890 (26)[a] (93)[b] 9.1[c]	.... 82 11.8	1550	2000
D	6.8	..	0 95	1380 (16)[a] (75)[b] 17.4[c]	2260 (16)[a] (62)[b] 14.7[c]	2880 (16)[a] (62)[b] 14.3[c]	.... 66[b] 15.5[c]	1700	2200
E	6.4	..	0.9	1300 (15)[a] (80)[b] 10.2[c]	2120 (15)[a] (94)[b] 7.5[c]	3040 (15)[a] (100)[b] 6.8[c]	.... 91[b] 8.1[c]	1900	2400
Average[d].	6.6	70	0.99	1420 (76)[b] 14.4[c]	2220 (82)[b] 11.3[c]	3020 (88)[b] 11.1[c]	.... 82[b] 12.2[c]	1600	2050

[a] Number of specimens tested.
[b] Per cent of total number of specimens falling within 20 per cent of the average.
[c] Mean variation, per cent.
[d] Averages not weighted for number of specimens.

TABLE 3.—COMPRESSION TESTS OF CONCRETE—JOB A.

Compression tests of 6 x 12-in. concrete cylinders made in field.
Aggregate: "Ready-mix" gravel from Port Jefferson, Long Island, furnished by Kittanning Sales Co. Portland cement was used.
Mix for 5 bag batches by damp and loose volume 1:6½; by weight 1:7; by dry and puddled volume 1:5½. Same quantity of aggregate used for 6 bag batches.
Samples selected from about 1 cu. yd. batch during discharge from chute.
Specimens cured in damp sand until test; tested damp.

Batch.	Date Sampled (1923).	Time of Mixing, sec.			Mixing Water, gallons per batch.	Slump in.	Flow, per cent.	Compressive Strength, lb. per sq. in.			Batch.	Date Sampled (1923).	Slump in.	Flow, per cent.	Compressive Strength, lb. per sq. in.
		Loading.	Mixing.	Discharging.				7 day.	28 day.	3 mo.					28 days.
CEMENT PER BATCH—5 BAGS.															
3	9-12	..	..	..	....	6 5	45	1020	1650	....	20	9-25	6.5	80	2210
								990	1860						2070
								1017	1780						2140
4	9-12	..	..	..	..	6.0	..	1140	2040	....	21	9-25	6 5	80	1805
								....	2210						1890
								1140	2120						1840
5	9-14	..	..	.	...	7 5	110	1490	2350	3970	22	9-25	5.25	80	2500
								1270	2240	3240					2120
								1380	2290	3600					2310
6	9-14	..	45	..	31.9	7.5	110	1790	2990	3410	23	9-25	4.5	50	2260
								1710	3040	3880					2480
								1750	3020	3650					2370
7	9-14	..	45	..	31.9	6.5	95	1580	2690	4250	24	9-25	4.5	50	1930
								1670	2870	2940					2000
								1620	2780	3600					1960
8	9-14	..	60	..	33.0	6.0	80	1420	2650	2840	25	9-26	7	..	1940
								1560	2120	3290					1930
								1490	2380	2930					1940
9	9-14	..	65	..	34.6	5.5	70	1440	2650	3380	26	9-26	7	120	2650
								1390	2270	3350					2660
								1410	2460	3360					2660
11	9-19	25	38	35	36.0	4.5	70	1470	2350	2900	27	9-26	6	80	2300
								1530	1790	2940					2310
								1500	2070	2920					2300
12	9-19	30	30	40	34.2	7.0	90	2180	2730	2780	28	9-29	4.5	55	1910
								2310	2900	4340					1810
								2310	2820	3560					1860
13	9-19	40	20	35	33.6	6.0	80	1680	2140	3360	29	10-2	5.0	60	1880
								1730	2540	2780					2090
								1700	2340	3070					1980
15	9-20	..	..	..	....	5.0	75	1570	2190	3150	30	10-2	4.0	50	1840
								1710	2370	3500					1930
								1640	2280	3320					1880
16	9-20	30	30	35	35.3	5.5	70	1810	2720	3330	31	10-2	6 0	70	1740
								1950	2510	3940					1610
								1880	2620	3640					1680

TABLE 3.—*Continued.*

CEMENT PER BATCH—5 BAGS.

Batch.	Date Sampled (1923).	Time of Mixing, sec. Loading.	Mixing.	Discharging.	Mixing Water, gallons per batch.	Slump in.	Flow, per cent.	Compressive Strength, lb per sq. in. 7 day	28 day.	3 mo.	Batch.	Date Sampled (1923).	Slump in.	Flow, per cent.	Compressive Strength, lb. per sq. in. 28 days.
17	9–20	30	30	30	31.2	6.0	70	1480	2180	2700	32	10-8	7.5	125	1900
								1350	2480	2520					2040
								1420	2330	2610					1970
18	9–20	30	20	40	30.4	6.5	80	1610	2360	2900	33	10-8	4	65	1680
								1710	2120	3220					1580
								1660	2240	3060					1650
19	9–20	..	..	..	....	6.5	80	1670	1900	3060	34	10–10	5 5	60	1570
								1650	2140	2850					1880
								1660	2020	3460					1725
37	10-15	26	23	40	34.5	...	..	1320	2000	2640	35	10–10	4.5	50	2370
								1270	1850	2350					2670
								1300	1920	2500					2520
38	10-15	21	39	44	34.5	7	..	975	1600	1900	36	10-10	6	100	1860
								975	1340	1850					1840
								975	1470	1880					1850
39	10-16	30	25	30	34.5	7	..	665	1380	2290	41	....	...	..	2070
								720	1980	2120					2000
								690	1530	2200					2040
40	10-16	30	25	30	34.5	7.75	..	1030	1785	2810	42	....	...	..	2060
								980	1650	2610					2010
								1000	1720	2710					2035
Grand average.........					33.7	6.1	80	1450	2130	3040					
Mean variation, per cent					....	...	..	19.7	14.4	17.8					

CEMENT PER BATCH—6 BAGS.

Batch.	Date Sampled (1923).	Loading.	Mixing.	Discharging.	Mixing Water, gallons per batch.	Slump in.	Flow, per cent.	7 day	28 day.	3 mo.
1	9–12	.	.	..	....	4.2	55	1810	3760	4450
								2880	4050	5000
								2340	3950	4725
2	9-12	..	..	..	....	7.0	50	2300	3210	4500
								2050	3440	4050
								2180	3320	4275
10	9-19	30	30	40	36.0	3.5	50	1910	2680	3440
								1750	2950	3620
								1830	2820	3530
14	9-19	25	25	35	35.3	6.0	80	1670	2570	3740
								2060	2540	3850
								1860	2560	3790
Grand average..........					35.6	5.2	58	2050	3160	4080
Mean variation, per cent.					....	...	..	12.6	14.7	10 4

TABLE 4.—COMPRESSION TESTS OF CONCRETE—JOB B.

Compression tests of 6 x 12-in. concrete cylinders made in field.
Aggregate: Sand and pebbles from Port Washington, Long Island, furnished by Lenox Sand and Gravel Co
Portland cement was used.
Mix by volume damp and loose 1:1.8:3.9; by dry weight 1:1.5:4.3; by volume dry and puddled 1:1.3:3.7
Samples selected from two-sack batch of concrete, near place of deposit.
Specimens cured in damp sand until test; tested damp.

Batch.	Date Sampled (1923).	Time of Mixing, sec.			Slump, in.	Compressive Strength, lb. per sq. in.		
		Loading.	Mixing.	Discharging.		7 day.	28 day.	3 mo.
1	8-6	10	65	15	8.5	1420	2800	3460
						1490	2580	3430
						1450	2590	3450
2	8-7	10	75	10	8.5	1510*	2260	2850
						1420*	2090	2780
						1460	2180	2820
3	8-7	..	..	..	8.7	1510*	2300	3080
						1530*	2250	2940
						1520	2280	3010
4	8-8	10	90	15	9.0	1500*	2140	3270
						1510*	2260	3000
						1500	2200	3135
5	8-8	..	..	..	8.7	1640*	1920	3000
						1630*	2060	2930
						1640	1990	2975
6	8-9	20	85	15	6.5	1770*	2620	3380
						1830*	2700	3380
						1800	2660	3380
7	8-9	20	55	20	7.2	1440*	2360	3540
						1350*	2380	3060
						1390	2370	3300
8	8-9	17	53	15	8.0	1710*	2510	....
						1800*	2590	3440
						1760	2550	3440
9	8-9	10	70	10	7.2	1450*	2110	3540
						1290*	2040	2860
						1370	2080	3200
10	8-14	12	80	20	8.0	1410	2360	3135
						1080	2300	2940
						1250	2330	3030
11	8-14	10	80	10	6.5	1560	2500	3380
						1650	2740	3360
						1600	2620	3370
13	8-14	17	110	15	8.0	1350	2420	3695
						2190	2480	3760
						1770	2450	3720
14	8-14	10	65	10	5.3	1630	2710	3380
						1680	2720	3230
						1660	2720	3300
Grand average					7.7	1550	2390	3230
Mean variation, per cent					...	10.0	8.2	7.4

* Age at test, 9 to 11 days.

TABLE 5.—COMPRESSION TESTS OF CONCRETE—JOB C.

Compression tests of 6 x 12-in. concrete cylinders made in field.
Aggregate: sand and pebbles from Port Washington, Long Island, furnished by Lenox Sand and Gravel Co.
Portland cement was used.
Samples selected from buggies by means of sampling box, during discharge of about ½ cu. yd. batch from mixer.
Specimens cured in damp sand until test; tested damp.

Batch.	Date Sampled (1923).	Time of Mixing, sec.	Slump, in.	Flow, per cent.	Compressive Strength, lb. per sq in.			Batch.	Date Sampled (1923).	Slump, in.	Flow, per cent.	Compressive Strength, lb. per sq. in.
					7 day.	28 day.	3 mo.					28 days.
					MIX BY VOLUME, DRY AND PUDDLED 1:1.9:4.0							
1	8–24	145	6.0	..	2110 2010 2060	2750 2800 2780	3170 3250 3210	..	...	..	..	....
2	8–24	80	5.75	..	1300 990 1140	2000 2240 2120	2610 2480 2540	..	...	..	..	....
3	8–24	..	6.0	..	1330 1350 1340	2350 2250 2300	2880 3110 2990	..	..	..	..	....
4	8–24	68	6.75	..	1380 1370 1370	2470 2250 2360	2780 2800 2790	..	...	..	..	....
					MIX BY VOLUME, DRY AND PUDDLED 1:2.1:4.2							
5	8–28	96	6.5	60	1420 1450 1440	2580 2360 2470	2820 2840 2830	14	9–5	..	..	2240 2210 2220
6	8–28	75	7.0	70	2000 1800 1900	2760 2870 2820	3370 3590 3480	15	9–5	..	..	1640 1790 1720
7	8–28	39	6.0	70	1540 1510 1520	2170 2040 2100	3080 2520 2800	16	9–5	7.0	..	1900 2140 1970
8	8–28	87	3.0	25	1590 1480 1540	2160 2460 2310	2460 2660 2560	17	9–5	..	..	1970 1960 1960
9	8–28	38	8.75	50	1360 1360 1350	2115 2260 2190	2830 2870 2850	18	9–5	..	..	1960 1890 1920
10	8–31	92	5.75	70	1560 1460 1510	2580 2840 2710	3620 3270 3440	19	9–7	..	..	2160 2140 2150
11	8–31	73	5.75	40	1330 1310 1320	2150 1770 1960	2500 2660 2580	20	9–7	..	..	2160 2270 2220
12	8–31	87	5.5	70	1190 1120 1160	1890 1900 1900	2860 2500 2680	21	9–7	..	..	2580 2780 2680
13	8–31	133	5.75	60	900 1070 980	1700 1760 1730	2660 2980 2820	22	9–7	..	..	2060 2090 2080
								23	9–7	..	..	2070 1810 1940
Grand average			5.9	60	1430	2180	2890					
Mean variation, per cent...			....	....	14.7	11.7	9.1					

TABLE 6.—COMPRESSION TESTS OF CONCRETE—JOB D.

Compression tests of 6 x 12-in. concrete cylinders made in field.

Aggregate: "Ready-mix" gravel from Port Jefferson, Long Island, furnished by Kittanning Sales Co. Portland cement was used.

Mix by volume: damp and loose, approximately 1:6.

Concrete samples selected from buggies by means of sampling box, during discharge, of about 1 cu. yd. batch from mixer.

Cylinders cured in damp sand until test; tested damp

Batch.	Date Sampled (1923).	Time of Mixing, sec.			Cement per Batch, bags.	Mixing Water, gallons per batch.	Slump, in.	Compressive Strength, lb. per sq. in.		
		Loading.	Mixing.	Discharging.				7 day.	28 day.	3 mo.
1	9-28	30	30	20	6	37.6	8	1110 1110	2220 2220	2610 2600
								1110	2220	2600
2	9-28	..	..	..	6	37.6	8	780 1040	1440 1540	1620 1900
								860	1490	1760
3	10-1	20	25	30	6	28.5	8.5	1310 1400	2400 2360	3020 2740
								1360	2380	2880
4	10-1	20	35	23	6	29.8	5.5	1660 1700	2990 2760	2970 3610
								1680	2875	3290
5	10-2	40	3	28	6	29.8	6	1610 1540	1990 2070	3160 3350
								1580	2030	3255
6	10-2	20	18	10	6	32.3	6.5	1300 1140	1960 2080	2140 2690
								1220	2020	2415
7	10-4	25	45	20	6	31.6	4.5	1920 1640	3000 2760	3640 3460
								1780	2880	3550
8	10-4	28	62	20	6	31.6	7	1490 1440	2220 2240	3370 3220
								1460	2230	3295
Grand average			....	....	....	32.4	6.8	1380	2260	2880
Mean variation, per cent			....	....	....	....	...	17.4	14.7	14.3

TABLE 7.—COMPRESSION TESTS OF CONCRETE—JOB E.

Compression tests of 6 x 12-in. concrete cylinders made in field.
Aggregate: "Ready-mix" gravel from Marlborough-on-the-Hudson, furnished by Rosoff Sand and Gravel Co.
Portland cement was used.
Concrete samples selected from forms for massive foundations.
Cylinders cured in damp sand until test; tested damp.

Sample	Date Sampled (1923).	Slump, in.	Compressive Strength, lb. per sq. in.		
			7 day.	28 day.	3 mo.
1	9-14.	...	980	2110	2850
			1060	2000	3290
			1010	2180	2440
			1020	2090	2860
2	9-17	5.5	1420	2400	3490
		5.2	1350	2660	3010
		6.4	1320	2540	3250
			1360	2530	3250
3	9-18	6.4	1550	2140	3400
4	....	7	1600	2210	3120
5	....	6	1430	2420	3320
6	9-19	6	1240	1800	2980
7	....	6.5	1320	1940	2790
8	....	6.5	1350	2050	2970
9	9-20	7.5	1300	2080	3120
10	....	6.2	1370	2010	2880
11	....	7.8	1240	2070	3090
Grand average	....	6.4	1300	2120	3060
Mean variation, per cent	....	...	10.2	.57	6.8

TABLE 8.—TESTS OF CONCRETE SAMPLED AT DIFFERENT POINTS—JOB A.

Compression tests of 6 x 12-in concrete cylinders.
Mix: about 1:5½ by volume, dry and puddled.
Aggregate: "Ready-mix" gravel from Port Jefferson, Long Island, furnished by Kittanning Sales Co.
Portland cement was used.
Three samples selected from a batch of approximately 1 cu. yd.; at mixer, discharge end of chute, and from forms.
Age at test 28 days.
Cylinders cured in damp sand until test; tested damp.

Batch.	Date Sampled (1923).	Slump, in.			Flow, per cent			Compressive Strength, lb. per sq. in.		
		Mixer.	Chute.	Forms.	Mixer.	Chute.	Forms.	Mixer.	Chute.	Forms.
28	9-29	..	4.5	5.0	..	55	65	1540	1910	1450
								1520	1810	1600
								1530	1860	1520
29	10-2	..	5.0	7.0	..	60	85	1910	1880	1840
								2070	2090	1630
								1990	1980	1740
30	10-2		4.0	6.0	..	50	80	1550	1840	1850
								1760	1930	1470
								1660	1880	1660
31	10-2	..	6.0	7.0	..	70	100	1510	1740	1760
								1710	1610	1940
								1610	1720	1850
Grand average			4.9	6.2	..	58	82	1700	1860	1690
Mean variation, per cent			...	...	..	..	..	9.1	5.4	9.2

TABLE 9.—SIEVE ANALYSES AND MOISTURE CONTENT OF AGGREGATE—JOB A.

Aggregate: "Ready-mix" gravel from Port Jefferson, Long Island.
Average weight of aggregate: damp and loose, 112 lb. per cu. ft.; dry and puddled, 128 lb.

Sample.	Date Sampled (1923).	Place Sampled.	Moisture, per cent by weight.	Amount Coarser than each Sieve, per cent by weight.										Fineness Modulus.*
				100	50	30	16	8	4	⅜	¾	1	1½	
1	9–9	Stock pile.........	...	99	93	80	72	63	54	43	31	26	5	5.40
2	"	" ".........	...	99	93	84	73	69	65	40	24	21	1	5.48
3	"	" ".........	...	99	96	83	73	68	63	40	26	22	0	5.48
4	"	" ".........	...	99	95	79	69	64	60	34	21	16	0	5.21
5	"	" ".........	...	99	94	84	75	67	61	46	30	24	10	5 66
6	"	" ".........	...	99	92	81	76	71	67	57	43	36	5	5 91
7	"	" ".........	2.3	99	96	86	78	74	69	42	27	21	3	5.74
10		At mixer	...	99	99	91	76	60	45	28	14	10	2	5.14
11		5 samples from one	...	99	98	90	75	62	49	32	17	11	0	5.22
12	9–12	batch collected at	...	100	99	91	79	69	57	35	17	12	3	5.50
13		different	...	100	99	96	89	85	79	61	27	16	2	6.38
14		intervals	...	99	99	91	80	71	64	50	32	21	9	5 95
15	9–12	Mixer.............	...	100	98	92	83	77	70	50	22	11	0	5.92
16	"	Mixer (Batch 5)...	...	99	95	84	73	64	56	31	10	4	0	5 12
17	"	" (" 6)...	1.7	99	95	84	74	66	61	37	14	6	0	5 30
18	"	" (" 7)...	1.6	99	93	81	74	66	59	42	23	16	1	5.38
19	"	" (" 8)...	1.7	99	93	80	72	62	53	35	19	4	0	5.13
20	"	" (" 9)...	1.6	99	97	90	80	71	65	47	28	21	2	5.79
21	"	Belt conveyor.....	...	99	98	86	69	55	42	24	10	5	0	4.83
22	"	" "	...	99	97	83	61	55	45	29	13	5	1	4.81
23	"	" "	...	100	97	82	64	50	42	27	13	9	4	4.79
24	"	" "	...	99	98	86	68	55	47	30	13	8	2	4.98
25	9–18	Stock pile.........	1.7	..	..	..	..	..	..	..	..	..	..	....
26	"	" ".........	1.8	..	..	..	..	..	..	..	..	..	..	....
27	9–19	Mixer (Batch 10)..	1.8	99	98	89	72	56	42	22	7	4	0	4.65
28	"	" (" 11)..	1.4	99	98	89	74	65	57	41	19	11	2	5.44
29	"	" (" 12)..	1.4	99	93	81	67	57	49	37	24	17	6	5.13
30	"	" (" 13)..	2.1	98	91	82	74	61	51	39	26	23	3	5.25
31	"	" (" 14)..	2.1	98	92	82	73	60	49	37	25	23	3	5.19
32	9–20	" (" 15)..	2.1	99	98	82	73	55	39	28	20	18	3	4.92
33	"	" (" 16)..	...	97	95	86	76	63	54	38	21	15	0	5.30
34	"	" (" 17)..	1.4	99	96	92	89	81	72	52	29	23	3	6.13
35	"	" (" 18)..	...	..	..	..	..	.	..	49	35	29	4	....
36	9–25	Belt conveyor.....	2.3	99	95	90	85	73	65	56	42	37	9	6.14
37	"	" "	1.9	99	96	91	87	78	72	64	50	42	5	6.42
38	"	Truck............	...	..	..	..	..	..	56	42	23	14	1	....
39	"	"	...	..	..	..	..	..	58	45	23	11	4	....
40	12–13	"	...	99	99	88	76	68	61	46	26	18	5	5.68
41	"	"	...	..	..	..	..	..	60	47	25	17	7	....
42	"	"	...	..	..	..	..	..	64	50	27	16	6	....
43	"	"	...	..	..	..	..	..	55	41	20	11	1	....
44	"	"	...	..	..	..	..	..	45	33	15	9	4	....
45	12–11	"	...	99	97	84	71	63	57	44	24	15	4	5.43
46	"	"	...	..	..	..	..	..	51	49	21	9	0	....
47	10–10	Mixer.............	1.5	99	95	85	76	67	61	47	26	17	3	5.59
48	10–19	"	...	99	97	88	77	68	61	45	22	14	2	5.59
49	"	"	1.0	99	98	90	80	71	64	48	26	15	3	5.79
50	12–11	Mixer (Batch 37)..	...	99	97	90	83	72	65	51	26	19	5	5.88
51	12–12	" (" 38)..	...	99	98	91	79	67	56	41	20	12	4	5.55
52	"	" (" 38)..	...	99	98	89	74	64	54	38	19	13	5	5.40
53	"	" (" 38)..	...	99	93	81	71	62	55	38	18	13	0	5.17
Average	....	....	1.7	99	96	86	75	65	57	41	23	16	3	5.46

* Sum of percentages in sieve analysis divided by 100, omitting the 1-in. sieve.

TABLE 10.—SIEVE ANALYSES AND MOISTURE CONTENT OF SAND AND PEBBLES—JOB B.

Sand and pebbles from Port Washington, Long Island.

Sample.	Date Sampled (1923).	Concrete, batch.	Moisture, per cent by weight.	Amount Coarser than each Sieve, per cent by weight.										Fineness Modulus.*
				100	50	30	16	8	4	⅜	¾	1	1½	
							SAND.							
..	8-5	1	...	96	81	40	21	9	3	0	..	..	..	2.50
3	8-6	..	...	96	81	39	19	8	2	0	..	..	..	2.45
5	8-7	2	...	95	82	36	16	7	2	0	..	..	..	2.38
7	8-8	4	5.8	97	87	40	18	7	2	0	..	..	..	2.51
10	8-8	..	3.3	..	..	..	..	..	..	..	..	..	..	....
11	8-9	..	3.5	..	..	..	..	..	..	..	..	..	..	....
13	8-9	6	...	96	81	40	20	6	1	0	..	..	..	2.44
14	8-9	..	3.5	..	..	..	..	..	..	..	..	..	..	....
16	8-9	..	...	95	80	38	17	6	2	0	..	..	..	2.38
18	8-9	8	4.6	96	82	42	20	8	2	0	..	..	..	2.50
20	8-9	9	...	96	78	38	11	6	2	0	..	..	..	2.31
22	8-14	10	3.6	97	81	38	17	8	2	0	..	..	..	2.43
24	8-14	11	3.5	96	81	38	16	7	2	0	..	..	..	2.40
26	8-14	12	3.8	95	81	40	18	7	2	0	..	..	..	2.43
27	8-14	13	3.5	96	81	40	15	7	2	0	..	..	..	2.41
29	8-14	14	3.6	96	82	43	20	8	2	0	..	.	..	2.51
30	8-14	..	4.0	..	..	..	..	..	..	..	..	..	..	....
31	8-14	..	3.5	..	..	..	..	..	..	..	..	..	..	....
32	8-14	..	4.4	..	..	..	..	..	.	..	..	..	..	....
33	8-14	..	3.7	..	..	..	..	..	..	..	..	..	..	....
Average..................			3.9	96	82	39	17	7	2	0	..	..	..	2.43
							PEBBLES.							
2	8-6	1	...	99	99	93	97	94	82	48	16	6	0	6.33
4	8-6	..	...	99	98	97	94	93	82	51	20	9	0	6.34
6	8-7	2	...	99	97	95	93	90	96	35	8	3	0	5.93
8	8-8	4	2.0	99	97	95	93	90	75	38	13	5	0	6.00
9	8-8	5	...	99	99	98	95	94	79	45	16	5	0	6.25
12	8:9	6	...	99	97	95	92	85	72	33	7	3	0	5.30
15	8-9	7	...	100	99	98	96	91	70	35	14	7	0	6.03
19	8-9	8	...	99	99	98	96	88	72	37	18	6	0	6.07
21	8-9	9	...	100	99	97	95	88	82	39	14	6	0	6.04
23	8-14	10	2.3	100	99	99	97	93	76	52	20	9	0	6.36
25	8-14	11	...	99	99	97	96	91	73	40	13	6	0	6.08
28	8-14	13	...	99	99	98	97	93	77	45	12	4	0	6.20
30	8-14	14	1.8	99	99	98	96	89	71	34	10	4	0	5.96
Average..................			2.0	99	98	97	95	91	76	41	14	6	0	6.11

* Sum of percentages in Sieve Analyses divided by 100, omitting the 1-in. sieve.

TABLE 11.—UNIT WEIGHT OF AGGREGATES—JOB B.

Unless otherwise noted, the unit weights were determined by puddling dry aggregates into a 1/6 cu. ft cylindrical measure, in accordance with the Standards of the American Society for Testing Materials. Unit weights of average samples of sand and pebbles; see Table 10 for sieve analyses.

Mixed Aggregate, per cent by weight.		Unit Weight, lb. per cu. ft.
Sand	Pebbles.	
DRY AND PUDDLED.		
100 (sand only)	0	108
60	40	117
50	50	121
41	59	123
33	67	124
28	72	124
23	77	121
17	83	120
13	87	118
0	100 (pebbles only)	110
DAMP AND LOOSE.		
Sand only (4 per cent moisture)		85
Pebbles only (2 per cent moisture)		107

TABLE 12.—DATA OF CALIBRATION OF MEASURING DEVICES—JOB B.

Quantities for batch were two sacks of cement plus one shovelful, two barrows of sand and four barrows of pebbles. The mixing water used for each batch was 87 lb. plus one or more of the additional quantities listed below. usually the medium

	Quantities.				
	1	2	3	4	Average.
Wheelbarrows for measuring sand:					
(*a*) Damp sand, lb.	148	155	164	164	159
(*b*) Dry sand, lb. (Weight of damp sand, less 4 per cent moisture)	142	149	157	157	151
(*c*) Sand, damp and loose, cu. ft.	1.75	1.83	1.94	1.94	1.87
(*d*) Sand, dry and puddled, cu. ft.	1.32	1.38	1.45	1.45	1.40
Wheelbarrows for measuring pebbles					
(*a*) Damp pebbles, lb.	228	212	210	....	217
(*b*) Dry pebbles, lb. (Weight of damp pebbles, less 4 per cent moisture)	223	203	206	....	212
(*c*) Pebbles, damp and loose, cu. ft.	2.12	1.98	1.97	....	2.02
(*a*) Pebbles, dry and puddled, cu. ft.	2.02	1.09	1.87	....	1.93
Water barrel					
(*a*) Initial quantity of water for each batch, lb.	......	......	......	......	87.0
(*b*) Water added to regulate consistency, lb.:					
Small amount	3.5	....	....	....	3.5
Medium amount	14.8	9.2	....	....	12.0
Large amount	35.0	....	....	....	35.0
Shovelful of cement, lb.	9.5	9.9	....	....	9.7

TABLE 13.—SIEVE ANALYSES AND MOISTURE CONTENT OF SAND AND PEBBLES—JOB C.

Sand and pebbles from Port Washington, Long Island.

Sample	Date Sampled (1923).	Concrete, batch.	Moisture, per cent by weight.	Amount Coarser than each Sieve, per cent by weight.										Fineness Modulus.*
				100	50	30	16	8	4	⅜	¾	1	1½	
SAND.														
3	8-21	None	...	94	74	34	17	7	1	0	..	.	..	2 27
6	8-21	None	2 2	95	80	44	21	8	1	0		.	..	2 49
7	8-21	None	3 3	96	81	45	21	7	1	0	..		..	2 51
9	8-24	1	2 5	97	78	35	17	7	1	0	.	.	..	2 37
11	8-24	2	3.8	96	77	38	18	7	1	0	.	..	.	2.38
13	8-24	3	4.7	96	79	39	19	8	1	0		..	..	2.45
15	8-24	4	4.5	97	77	37	19	7	1	0	..	..	..	2 38
22	8-28	5	4.2	96	78	35	15	6	1	0	.	..	..	2.31
24	8-28	6	4.5	96	78	36	16	7	1	0	..	..	..	2.34
28	8-28	8	4.7	95	80	44	24	9	1	0	..	..	..	2.53
30	8-28	9	6.2	96	80	47	24	9	2	0	..	..	..	2.58
32	8-31	10	6 0	96	82	45	22	8	2	0	..	..	..	2.55
34	8-31	11	6.0	96	80	43	22	7	1	0	..	..	..	2.49
36	8-31	12	6.2	96	79	45	26	10	3	0	..	..	..	2.59
38	8-31	13	7.3	97	80	44	22	8	2	0	..	..	..	2.53
40	9-5	14	...	98	87	41	15	6	1	0	..	.	..	2.48
42	9-5	14	...	96	80	35	14	6	1	0	..	.	..	2 32
44	9-5	15	...	95	78	41	19	6	1	0	..	..	.	2.40
4	8-21	None	...	97	82	44	23	5	2	0	..	..	..	2.53
5	8-21	None	5.1	96	82	48	23	6	0	0	.	..	..	2.55
8	8-21	None	...	96	81	44	21	7	1	0	..	..	..	2.50
..	8-21	None	...	96	80	42	33	6	2	0	.	..	..	2 49
Average			4.7	96	80	41	20	7	1	0	.	..	..	2.45
PEBBLES.														
1	8-21	None	...	99	99	99	98	92	69	34	11	5	0	6.01
2	8-21	None	...	99	99	99	94	87	67	33	11	5	0	5.87
10	8-27	1	2.9	99	99	98	96	92	77	48	20	9	0	6 29
12	8-27	2	2.1	99	98	96	95	92	85	40	7	3	0	6.12
14	8-27	3	1.8	100	99	99	99	97	89	46	14	3	0	6.45
16	8-27	4	1.4	99	97	95	94	92	86	71	23	3	0	6.57
18	8-27	None	...	99	99	97	96	94	88	51	14	3	0	6.42
20	8-27	None	...	..	..	..	..	..	84	48	10	3	0	6.30†
21	8-27	None	...	..	..	..	..	..	81	39	9	2	0	....
23	8-27	5	2.7	99	99	98	97	95	90	47	9	1	0	6.34
25	8-27	6	3.0	99	98	96	94	91	83	40	8	2	0	6.09
27	8-27	7	3.4	99	97	95	92	89	82	39	7	1	0	6.00
29	8-23	8	2.4	99	98	97	96	95	89	46	9	2	0	6.29
31	8-23	9	3.0	99	99	98	96	95	89	53	14	4	0	5.48
33	9-7	10	...	99	98	96	93	89	77	48	23	12	0	6.23
35	9-7	11	...	99	98	96	94	88	74	41	17	7	0	6.07
37	9-7	12	4.1	99	98	95	93	89	77	46	19	8	0	6.16
39	9-7	13	...	99	99	97	96	89	76	42	16	8	0	6.14
41	9-5	14	...	99	99	98	97	95	83	53	24	12	0	6.48
43	9-5	15	...	99	99	98	96	90	68	36	17	8	0	6.05
45	9-5	..	...	99	99	98	97	93	77	45	12	4	0	6.20
Average			2.7	99	99	97	95	92	81	45	138	5	..	6.18

* Sum of percentages in sieve analyses, divided by 100, omitting the 1-in. sieve.
† Fineness modulus estimated from average analyses of particles finer than No. 4 sieve.

TABLE 14.—UNIT WEIGHT OF AGGREGATE—JOB C.

Unit weights of mixed aggregates determined in ½ cu. ft. measure in accordance with Standards of American Society for Testing Materials.
Unit weights of average samples of sand and pebbles; see Table XIII for sieve analyses.

Mixed Aggregate, per cent by weight.		Unit Weight, lb. per cu. ft.				
Sand.	Pebbles.	1	2	3	4	Average.
100 (sand only)	0	108	108	108	108	108
60	40	122	121	120	120	121
50	50	124	124	123	124	124
40	60	127	126	127	126	126
35	65	128	127	127	128	128
33	67	129	128	126	128	128
30	70	128	128	125	127	127
25	75	126	125	123	125	125
20	80	124	122	121	122	122
0	100 (pebbles only)	111	112	110	111	111

TABLE 15.—SIEVE ANALYSIS OF AGGREGATES—JOB D.

Aggregate: "Ready-mix" gravel from Port Jefferson, Long Island.

Sample.	Date Sampled (1923).	Place Sampled.	Amount Coarser than each Sieve, per cent by weight.										Fineness Modulus.*
			100	50	30	16	8	4	⅜	¾	1	1½	
1	9–17	Stock pile	100	99	93	82	73	65	51	34	..	11	6.08
2	9–17	Stock pile	100	99	89	81	72	63	48	26	..	5	5.84
3	9–17	Stock pile	100	99	92	76	66	58	39	19	..	4	5.53
4	9–17	Stock pile	99	98	90	67	55	44	31	19	..	4	5.07
4	9–17	Stock pile	100	99	94	82	59	46	32	16	..	1	5.29
5	9–17	Stock pile	99	90	89	71	57	46	31	12	..	2	5 06
6	9–17	Stock pile	99	98	89	77	56	52	38	19	..	4	5.32
7	10–1	Mixer (Batch 3)	100	99	89	75	66	58	46	25	16	5	5.63
8	10–2	" (" 4)	100	99	91	77	65	54	39	20	12	2	5.47
9	10–2	" (" 5)	99	98	87	70	59	51	38	20	12	2	5.24
10	10–2	" (" 6)	99	98	88	73	64	56	41	22	13	1	5.42
11	10–4	" (" 7)	99	98	87	71	62	55	41	20	..	3	5.36
12	10–4	" (" 8)	99	97	85	71	63	57	45	26	..	5	5.48
Average			99	99	90	75	63	54	40	21	13	3	5.45

* Sum of percentages in sieve analysis, divided by 100, omitting the 1-in. sieve.

TABLE 16.—SIEVE ANALYSIS OF AGGREGATE—JOB E.

Aggregate: "Ready-mix" gravel from Marlborough-on-the-Hudson.

Sample.	Date Sampled (1923).	Scow Sampled.	Amount Coarser than each Sieve, per cent by weight.										Fineness Modulus.*
			100	50	30	16	8	4	⅜	¾	1	1½	
1	9–14	Bishop, 18 ft. from stern.....	98	97	93	87	79	72	49	14	3	0	5.89
2	" "	" 20 " " "	98	91	79	64	49	40	29	12	3	0	4.62
3	" "	" 6 " " top, 25 ft. from stern..............	99	96	91	82	72	63	51	22	5	0	5.76
4	" "	Bishop, near center, 18 in. down	98	94	84	69	55	47	39	18	6	0	5.01
5	" "	" bow, 18 in. down......	99	94	87	76	66	59	41	11	3	0	5.33
6	9–17	Marlboro, No. 10............	99	96	89	79	67	54	29	6	1	0	5.19
7	" "	" " "............	99	98	94	88	79	70	43	10	4	0	5.81
8	" "	" " "............	98	95	88	78	67	60	41	10	3	0	5.37
9	" "	" " "............	98	95	88	77	64	52	37	16	4	0	5.27
10	" "	" " "............	98	94	84	68	49	35	24	9	3	0	4.61
11	9–18	Wm. Howland...............	98	90	80	69	57	53	40	16	7	0	5.03
12	" "	" "	98	94	85	72	58	47	38	15	5	0	5.07
13	9–23	Eads..........................	98	94	85	73	63	55	44	20	7	0	5.32
14	" "	"	99	95	89	80	70	62	40	14	4	0	5.49
15	" "	"	98	96	88	81	72	65	43	11	2	0	5.54
16	9–25	D. W. Mack.................	99	98	94	87	80	72	51	12	3	0	5.93
17	" "	" " "	98	93	82	70	55	46	36	17	6	0	4.97
18	" "	" " "	99	94	85	73	59	50	37	14	5	0	5.11
Average..............................			99	95	87	76	65	56	40	14	4	0	5.30

* Sum of percentages in sieve analysis, divided by 100, omitting the 1-in. sieve.

TABLE 17.—MISCELLANEOUS LABORATORY TESTS OF CEMENT.

Tests made in accordance with Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials.
Each value is the average of two tests made on different days.

Cement Lot.	Brand of Cement.	Job.	Date (1923).			Residue on 200 Mesh Sieve.	Normal Consistency.	Time of Setting.								Soundness Test over Boiling Water.
								Vicat Needle.				Gillmore Needle.				
			Sampled.	Received.	Tested.			Initial, h.	m.	Final, h.	m.	Initial, h.	m.	Final, h.	m.	
7049–51	Penna...	A	9–14, 19, 20	10–2	10–26	16.6	22.7	3	35	8	25	5	40	9	20	O.K.
7145	Penna...	A	11–16	11–26	12–20	16.4	24.5	3	20	9	45	5	30	11	05	O.K.
6966	Penna...	B	7–7, 9, 14	7–18	10–26	19.4	23.5	3	25	8	00	5	45	9	10	O.K.
6986	Whitehall	C	7–24, 28, 31	9–10	10–26	19.8	23.0	3	25	7	50	5	40	8	40	O.K.
7098	Alpha...	D	10–31, 11–1	11–2	12–20	18.8	23.0	3	50	7	50	6	30	9	00	O.K.
7044	Atlas....	E	9–20	9–29	10–26	19.0	32.5	3	20	8	20	5	40	9	20	O.K.

TABLE 18.—CONCRETE AND MORTAR TESTS OF CEMENT IN LABORATORY.

Strength tests of 1–3 standard sand mortar briquets, and 2 x 4 in.-cylinders.
Compression tests of 6 x 12-in. concrete cylinders.
Aggregate for concrete: sand and pebbles from Elgin, Ill., graded 0–1½ in. (m = 5.51)
Relative consistency of concrete, 1.00.
Cement assumed to weigh 94 lb. per cu. ft.
In comparing strength values of concrete in this table with values obtained from the field tests it should be borne in mind that the laboratory cylinders were of drier consistency than those made in the field and therefore would be expected to give higher strengths at all ages.

Each value is the average of five tests unless otherwise noted.

Cement Lot.	Tensile Strength, lb. per sq. in.			Compressive Strength, lb. per sq. in.					
	1:3 Standard Sand Briquets.			1:3 Standard Sand.			1:5 Concrete.		
				2 x 4-in. Cylinders.			6 x 12-in. Cylinders.		
	7 days.	28 days.	3 mo.	7 days.	28 days.	3 ms.	7 days.	28 days.	3 mo.
7049–51	360	420	415	2080	3370	4110	2300	3640	4640
7145	365*	440*	535	2580*	3440*	4200	....	....	....
6966	280	395	440	1550	2020	2960	1720	2680	3370
6986	360	430	450	1820	3090	3700	2140	3500	4230
7098	295	395	470	1760	2700	3360	1570*	2680	4120
7044	325	370	445	2050	3200	3800	....	....	....

* Average of four tests.

DISCUSSION.

C. M. Chapman.—This investigation was undertaken, I believe, at the instigation of certain contractors who felt that the recommendations of the Joint Committee were impractical of operation on the job, that the newly proposed method of specifying concrete, not by giving definite proportions for the mix—but by stating the strength of the concrete which must be secured,—could not be satisfactorily worked out on the job.

To ascertain the practicability of the new method this investigation was undertaken. It would be interesting to know whether or not the investigation resulted in convincing the contractors that the method is workable. This paper gives the result from the technical side, but what did the contractors think of it? If they were to submit a report to us, what would it be?

Stanton Walker.—We have studiously avoided that question.

J. G. Ahlers.—It was not up to us. We are reporting facts. In my personal opinion, I feel just as strongly about that as I did three years ago. It is not a contractor's function to specify what shall be done, and that is not the purpose of this report. We have only made conclusions based on facts. I believe the Joint Committee is gathering a lot of these facts. We will let things carry on on that.

A study of the proportions and materials carried out in the light of knowledge now available will enable the engineer to estimate within 20 per cent of the average strength of the concrete when made in accordance with the general practices followed on these jobs. My personal opinion is that it will be very difficult to get contractors' organizations, timekeepers and others, to take the time and concentrated effort to control the concrete properly. This is the function of some organization outside of that of the contractors. For a firm to guarantee the strength of concrete in a structure would be utter folly, I would never and I do not believe other contractors would sign a contract that might make them tear the whole building down if certain specimens were not up to the strength guaranteed. I feel, furthermore, that we are making much better concrete than we are usually given credit for; therefore I do not believe the engineers should worry much about the guarantee.

A. N. Talbot.—I would like to ask a question. Reference is made here in the conclusions to certain relations. Were other mixtures experimented with than the particular ones named in the text?

Stanton Walker.—No other mixtures than the ones reported in the paper were experimented with during the investigation carried out in co-operation with the group of New York contractors. During the Camden investigation, however, which will be mentioned later during this session, tests were made using concrete of a wide range in cement content and proportions of sand to pebbles.

PROF. TALBOT.—First, I want to say something in commendation of this investigation as a method of learning whether concrete on the job can be controlled within limits that seem reasonable to give uniform strength. It seems to me that this is a very important report which has been given. It tells what is possible with the co-operation of the contractor and engineer or other organization. Whether such operations will cost too much or not, whether owners will be willing to pay the cost, or even whether it is necessary to have such an organization is another question. I shall not be surprised if we come to this control in spite of the fact that it may require more time and trouble on the part of the contractor. I want then personally to express my appreciation of this investigation.

Now, having said that, I shall have to give the opinion that some of the conclusions noted do not seem to me to be borne out by the results of the investigation given in the paper. I wonder why the particular combinations of materials were used. I have not had a chance, since I received the paper, to look it over very carefully. I see that separate values of the fine and coarse aggregate are given for Job B and Job C. These convey some idea of the kind of material used. From tests made in various laboratories, it appears that a concrete with less coarse material in it and a larger proportion of sand, using the same total amount of cement, would give strengths that are fairly comparable with the value obtained with this particular mix, not very much different from it, especially in the case of Job B. Such a mix would, it seems to me, be more workable on a job of reinforced-concrete work. I should think it would have been advisable to make tests to see if another mixture could not be placed more easily, to see if less water could not have been used with the same degree of workability. There is a further matter connected with these strength tests. The specimens were, of course, made by the standard method of making test specimens. With a coarse mix (one having a large proportion of coarse aggregate), there is a good deal of tamping done in getting the concrete into the mold. If, instead of using the standard method, a method had been used similar to that by which the concrete is put into the forms, throwing it in with a limited amount of tamping, the strength of the coarse mix would have been smaller proportionately than that found with a finer mix. I do not know what it would be with the particular mix used, but with a similar amount of coarse material the strength of the concrete placed as it is ordinarily put in work might for the coarser mixture be only 65 or 70 per cent of the strength obtained by the standard method of making the test pieces, while for the finer mixture it would much more nearly reach that of the standard laboratory test specimens.

As to the conclusions, I judge that some of these are not conclusions of the research, that they are rather the judgment of the authors. I refer, for example, to No. 5, "The fineness modulus of an accurate yardstick for measuring the grading of aggregate for the purpose of determin-

ing the economical proportions of fine to coarse." I cannot agree that has been proved by this paper. I should expect that economical proportions, granting there is not too much difference in the cost of the sand and gravel, might vary largely from that used. Also No. 6, "the maximum permissible values of fineness modulus given in Table 3, Bulletin 1 of the Structural Materials Research Laboratory may be satisfactorily applied to building construction." It may be granted that "these values of fineness modulus produce concrete that works somewhat more harshly than that to which the mechanics are accustomed, and in certain cases a campaign of education may be desirable before the maximum values of Table 3 are used. Reduction of these maximum values by 0.25 produces concrete which is easily worked into the forms in building construction." That, as I understand it, is merely a question of the practicability of using such a mixture, and not one of whether it is the best or most economical mixture. "Concrete designed under the conditions encountered in this investigation gave strengths well above the value in the Joint Committee tables of proportions." If what is referred to are the tables of proportions given in the report of the Joint Committee for sands from zero to 28, zero to 8, etc., I do not see anything in this investigation that shows whether those proportions are generally applicable. It may be that the tables are applicable in the two cases, Job B and Job C, though in looking at the table, it does not so appear. In referring to the Joint Committee tables of proportions, as I said in this convention two years ago, it seems to me that these tables, in various parts at least, are not fully reliable, and it would be unfortunate if such tables were put out as representing the strengths of such mixtures, particularly if specifications were to be based upon them. I am sorry to make these remarks on these conclusions, but I do it not because I am always a heretic but because I doubt some of the things that have been said over and over again and I fear that unless doubt is thrown on them some may come to think that they are generally true.

MR. WALKER.—Professor Talbót feels that a more easily handled concrete would have been obtained if a greater proportion of sand had been used. On none of the five jobs investigated was the sand content low enough to produce concrete that was not satisfactorily workable. Even on the jobs on which "Ready-Mix" was used only occasional batches were sufficiently deficient in sand to make them at all harsh working.

Professor Talbot has suggested that it would have been advisable to have made tests to determine the most economical proportions of fine to coarse aggregate, taking into account the ease of placing the different concretes. It is unfortunate that we were not able to follow up, with tests, the numerous sidelights that the investigation suggested. However, the chief object of the investigation was to determine the strength and uniformity of concrete placed under job conditions, and in the limited time at our disposal it was not possible to do much work aside from our outlined procedure. Also, we went on each job with the understanding

that we would in no way interfere with the usual operations, and it would have been impossible to have carried out work of the nature suggested without in some measure overstepping this condition. However, as stated above, as I look back on this work I do not see that any important gain in workability could have been obtained by increasing the proportion of sand.

We arrived at the conclusion that "The fineness modulus is an accurate "yardstick" for measuring the grading of aggregate for the purpose of determining the economical proportions of fine to coarse," and that "The maximum permissible values of fineness modulus given in Table 3, Bulletin 1, of the Structural Materials Research Laboratory may be satisfactorily applied to building construction," etc., from the fact that we were able to estimate, from calculations based on the fineness modulus, the proportions containing the minimum quantity of sand to produce satisfactorily workable concrete. Barring wide differences in the cost of fine and coarse aggregates, it seems that further demonstration is not needed to conclude that this is the most economical mix.

Our conclusion No. 7 is poorly worded and needs revision. It reads: "Concrete designed under the conditions encountered in this investigation gave strengths well above the values in the Joint Committee Tables of Proportions." It should read, "Concrete designed under the conditions encountered in this investigation gave strengths well above the values computed on the same basis as the Joint Committee Tables of Proportions."

The rewording alone, I think, answers Professor Talbot's question. In other words, if the strength of the concrete tested in this investigation is estimated from the same basis as the strengths in the Joint Committee Tables of Proportions we arrive at the above conclusion. As stated in the paper the Joint Committee Tables were based on the water-ratio-strength-relation of

$$S = \frac{14000}{9^x}$$

where S = compressive strength in lb. per sq. in. at 28 days,
x = water-ratio; ratio of volume of water to volume of cement (an exponent)

In general, the average strengths could be more closely estimated from a similar equation, which gives somewhat higher values of strength for a given water-ratio. This equation is:

$$S = \frac{14000}{7^x}$$

The first equation could be generally depended upon to give a close approximation to the minimum strengths.

PROFESSOR TALBOT.—May I say one word more, for fear I may be misunderstood in my remarks concerning the strengths of specimens as made by standard laboratory practice? I did not refer to whether the speci-

mens were representative of the absolute strengths of the concrete in the work. I had in mind that this method of making specimens does not give a real comparison of the strength of the coarse mix as it would be put into the work and that of the finer mix as it would be put into the work.

I would like to say also that it seems to me another conclusion ought to be added, that one cause of the uniformity of results that was obtained was the knowledge and the judgment of Mr. Walker and others who had the work in charge, even more than the particular means or the methods that were used for determining the mixtures. Their supervision, their judgment, their knowledge of the materials were, it seems to me, very important in getting the results.

H. C. BOYDEN.—May I offer a suggestion with regard to Professor Talbot's discussion, in which he comments on the fact that the average results of tests show uniformly higher than the values that would be expected by the use of the Joint Committee table? As this statement has been made in a number of cases in different parts of the country, it appears to me that the facts regarding this should be emphasized. The Joint Committee table and Bulletin No. 9 of the Structural Materials Research Laboratory are based on the equation S equals 14000 over 9^x, in which x is the water-cement ratio. With a ratio of 1.00 this would mean a value for S of 1555 lb. In Bulletin No. 1 of the Structural Materials Research Laboratory the equation used is, S equals 14000 over 7^x, giving a value for S, with the same water cement ratio, of 2000 lb., or an increase of 28.6 per cent. This means that the actual average results will be found much closer in accordance with the curve S equals 14000 over 7^x, as given in Bulletin No. 1, which is an average curve. With these facts realized, it may be that new understanding of the test results may be had and that the Joint Committee table and Bulletin No. 9 may be used with a greater accuracy.

CONTROL OF CONCRETE FOR THE UNIVERSITY OF ILLINOIS STADIUM.

By W. A. Slater * and R. L. Brown.*

The Structure.—The Illinois Memorial Football Stadium at Urbana-Champaign, Illinois, is a structure in which the footings, the ramps, the treads and risers carrying the seats, and some of the columns are of concrete. The framework and columns carrying the treads and risers for the

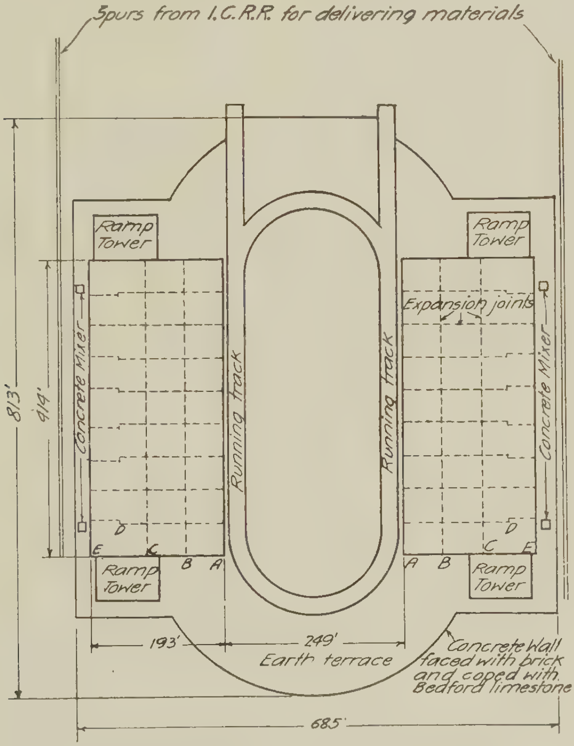


FIG. 1.—GENERAL PLAN OF STRUCTURE.

main stand and the balconies are of structural steel, and the walls above the basement of the ramp towers are of brick. The base course, trim, and memorial columns are of Bedford limestone.

The plans provide for a U-shaped structure when complete. Only the portions forming the sides of the U are now under contract, and these

* Urbana, Ill.

portions have a seating capacity of about 57,000 spectators. The general plan of the structure is as shown in Fig. 1.

It will be seen from this figure that the concrete was distributed over an area whose width is 348 ft., and whose length is 813 ft. The concrete materials were distributed to the site over spurs from the Illinois Central Railroad as shown in Fig. 1.

The structure was placed under contract in 1922, and most of the footings and substructure walls were built in the fall and winter of 1922-23.

The steel work and also the concrete of the superstructure for all of the seat stands and the two north ramp towers were erected and placed during the spring, summer and fall of 1923. The concrete of the two south ramp towers and considerable of the brick-and-stone work remain yet to be placed. Holabird & Roche of Chicago are the architects and engineers. English Brothers of Champaign, Ill., are the general contractors. M. B. Augur is the superintendent for the architects. Mr. Slater served as owners' representative on the concrete work, beginning about April 1, 1923. Mr. Brown carried out the preliminary laboratory studies of the concrete under the general oversight of Mr. Slater. He also assisted in the making of the control specimens and tests in the field and directed the testing of the cylinders for strength. The University of Illinois contributed a considerable part of Mr. Brown's time on this work and the laboratory facilities for making the tests.

Materials and Equipment.—Universal portland cement tested by the Robert W. Hunt Co., crushed limestone from the Lehigh Stone Co., of Kankakee, Illinois, and sand from the Lincoln Sand and Gravel Co., of Lincoln, Illinois, have been used throughout the work. The cement was furnished in cloth bags and was stored in wooden buildings, one near each mixing plant. Considerable of this cement was not used until it was four or five months old and a small quantity of it was close to a year old before it was used. Laboratory tests on concrete made from cement near the surface of the bag which appeared to have been damaged by moisture, gave a strength about the same as when undamaged cement from the center of the bag was used.

Frequent sieve analyses were made of the crushed stone used as coarse aggregate. Although under the specifications any aggregate having more than 5 per cent passing a 4-mesh sieve could have been rejected, this right was not exercised unless the material finer than a No. 48 (Tyler) sieve exceeded about 3 per cent.

Occasional sieve analyses and colorimetric tests (See A. S. T. M. Specification C-40-22 adopted in 1922) were made on the sand. It was quite uniform in gradation and gave no color within 24 hours in the colorimetric test. Tables 1 and 2 give representative sieve analyses of the sand and the crushed stone. For the superstructure a concrete mixer of one-half cubic yard capacity was placed approximately at each of the four corners of the structure as shown in Fig. 1. Three of the mixers were of Koehring manufacture. These mixers were equipped with water tanks which auto-

matically measured the water to added in the mixer. The measuring device on two of them were calibrated and the results of the calibration are given in Fig. 2. The agreement between the curves for the two mixers shown in this figure is sufficiently good to warrant the belief that the water was measured quite accurately.

For the outlying portions of the work smaller portable mixers were used.

TABLE 1.—SIEVE ANALYSIS OF SAND.

Sample No.	Per Cent Passing Sieve No.*					
	4 (0.185)	8 (0.093)	14 (0.046)	28 (0.023)	48 (0.0116)	100 (0.0058)
2	95.4	83.8	68.4	48.2	11.8	1.8
12	92.6	79.6	63.8	43.0	8.3	1.7
22	95.4	78.5	59.7	40.0	13.2	2.7
29	97.9	87.9	75.3	58.8	17.1	2.9
44	99.7	92.8	76.5	58.6	23.0	4.3
Average	96.2	84.5	68.7	49.7	14.5	2.7

* Numbers in parentheses are sizes of sieve openings in inches.

TABLE 2.—SIEVE ANALYSES OF STONE.

Sample No.	Per cent Passing Sieve No.*					
	¾ in.	½ in.	⅜ in.	4 (0.185)	14 (0.046)	48 (0.0116)
1	86.5	37.3	11.6	4.2	2.3	1.8
11	86.4	49.8	18.0	13.2	4.0	2.9
21	97.5	66.8	35.7	14.5	4.5	3.3
28	84.8	39.4	14.0	5.5	2.9	2.3
32	90.5	38.5	12.5	4.9	2.8	2.4
38	98.2	59.3	24.7	8.4	3.9	3.2
47	97.3	57.3	17.9	5.2	3.3	2.9
55	91.3	47.2	15.9	3.5	2.3	2.1
62	90.3	44.1	16.4	4.1	2.9	2.5
Average	91.4	48.9	18.5	7.1	3.2	2.6

* Numbers in parentheses are sizes of sieve openings in inches.

The materials were measured in wheelbarrows. Occasionally the measurement was verified by filling a wheelbarrow to the proper height using measuring boxes of known capacity.

Measurements of the hoppers used on the industrial track to carry concrete from the mixer gave the volumes of the hoppers when filled to various heights. Measurements of the depth of the concrete below the top of the hopper were made for frequent batches and this furnished a check on the measurement of materials and on the yield of concrete per barrel of cement.

The concrete of the main stand was poured in three lifts. The first lift was the portion between the running track and the expansion joint at line *B*, Fig. 1. The second lift was the portion between the expansion joints marked B and C, and the third lift was from expansion joint C to the top of the main stand.

For pouring the first lift industrial tracks extended from the mixer to the upper line of that lift and from there parallel to the line B marked expansion joint. In pouring any section the concrete was brought in one-half cubic yard hoppers to the upper part of the section in question and from there was chuted to the desired position in the section. For the two upper lifts the industrial tracks extended along the line C and the

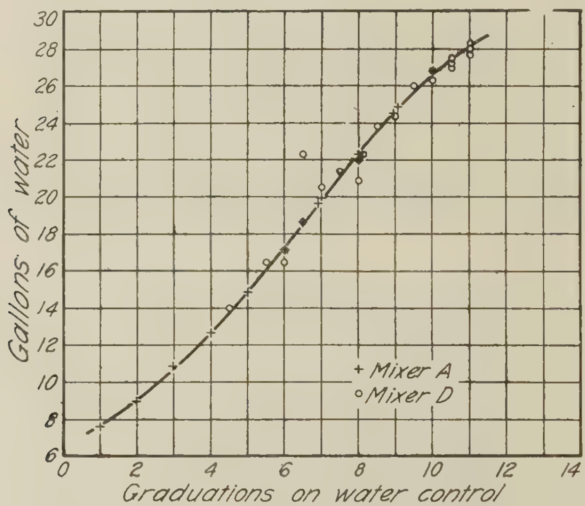


FIG. 2.—CALIBRATION CURVE FOR WATER-MEASURING DEVICES.

chutes from that line to the place of deposit of the concrete. For depositing the middle lift the maximum length of chute was a little more than 100 ft. The slope of the chutes was approximately one vertical to three horizontal.

The balcony was poured all in one lift. Here the slope was approximately one vertical to one and two-thirds horizontal. This gave chutes nearly a hundred feet long. The very first concrete to come down the clean steel chutes in pouring the balcony segregated, but after the first half of the first batch of each pouring, no further segregation took place.

Design of Concrete.—The specifications call for 1 : 2 : 4 concrete, and presumably the design was based on strength of 2,000 lb. per sq. in. for the concrete. It was agreed in advance of pouring the seats, however, that if experimental studies indicated that changes in the proportions were neces-

sary, in order to produce the desired strength, density, or workability, such changes would be made and the contractor would be reimbursed for any excess of cement required. A considerable number of specimens were made in the concrete laboratory of the University of Illinois Engineering Experiment Station using different proportions, different coarse aggregates, and different maximum sizes of coarse aggregate. An experimental section of the seat stand also was built to determine whether concrete of the proportions arrived at could be placed satisfactorily. Batches of 4.5 cu. ft. of concrete of a number of the mixes used in the laboratory were tried out. A 1: 2.5: 3.33 mix by loose volume with a maximum size of coarse aggregate of ⅝-in. was thus arrived at. This is approximately the same mix as was arrived at by the method proposed by Committee C-6 in this year's report. As delivered the coarse aggregate ran somewhat larger than ⅝ in.

TABLE 3.—COMPARISON OF CEMENT REQUIREMENTS OF TWO CONCRETES.

Batch No.	Barrels Cement per cu. yd. Concrete.	
	1:2:4	1:2.5:3.33
1	1.50	1.50
2	1.52	1.49
3	1.48	1.50
4	1.49	1.50
5	1.46	1.46
Average	1.49	1.49

The water-cement ratio for the laboratory batches which formed the basis for the mix finally used was 1.15 and the average slump was 7.5 in. The strength at 7 days for these specimens averaged 1545 lb. per sq. in.

After this mix had been in use for sometime a test was made to determine the amount of cement per cubic yard for both this concrete and the 1: 2: 4 concrete specified. In determining the cement per cubic yard of concrete the aggregates were measured loose with measuring boxes. The water was measured with the automatic device previously mentioned. Three bags of cement were used in each batch. The volume of concrete per batch was determined by measuring the depth to the surface of the concrete in the hopper. Five consecutive batches of one-half cubic yard each gave the quantity of cement shown in Table 3. It will be seen that no greater amount of cement was required for the 1: 2.5: 3.33 than for the 1: 2: 4 concrete.

Inspection and Control Tests.—During the placing of concrete in the seat stands an inspector from the architect's office was detailed to each side of the structure. When concreting was in progress on one side the inspector for the other side was free to inspect the placing of steel and the construction of forms. In addition to this the owner's representative

and the architect's superintendent had general supervision of all concreting operations. The inspectors reported to the architect's superintendent. Specimens for strength tests were 6 x 12-in. cylinders. These were taken at intervals throughout the work, sometimes not more than two a day, and at other times as many as eight in a day. These specimens were generally taken from the end of the chute just as the concrete arrived at the place where it was to be used in the forms. The entire flow of the

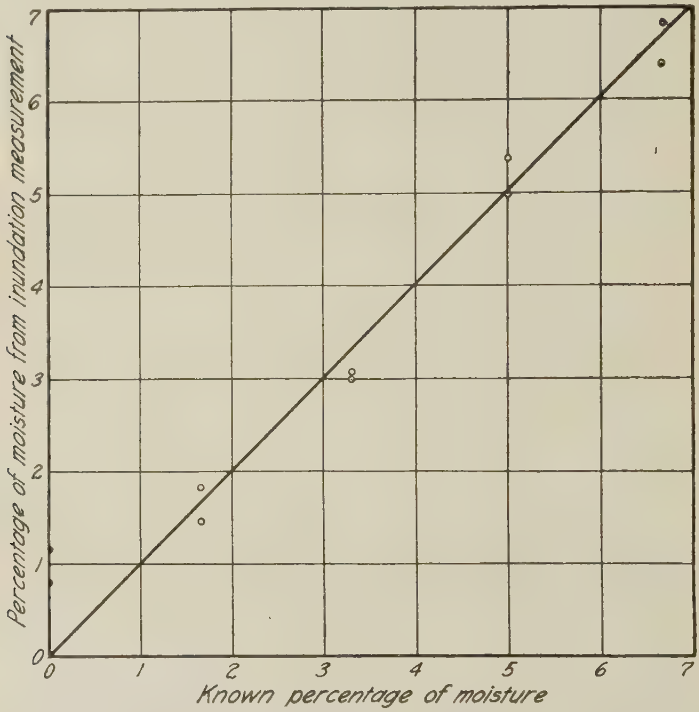


FIG. 3.—VERIFICATION OF INUNDATION METHOD OF DETERMINING MOISTURE IN SAND.

chute was collected for a length of time sufficient to fill the sampling bucket. In a few cases the concrete was taken from the forms. In general a slump test was made from the concrete so selected and then a cylinder was made from the same concrete. Six by twelve-in. "Cilicon" paper molds manufactured by the Standard Concrete Apparatus Company, of Washington, D. C., were used. These molds are of heavy paper, heavily waxed, entirely impervious, and satisfactory as cylinder molds.

Moisture in Sand.—The moisture in the sand was determined at frequent intervals by the inundation method. This method is based upon the

fact that if moist sand in a known quantity be poured into a vessel of known volume already containing water without entraining air voids the weight of the dry sand may be computed accurately from the specific gravity of the sand and the known volume of the water. Actually some air will be entrained* and this will introduce a slight error. If, however, for the specific gravity the apparent specific gravity of the sand and the included air voids be used the error will be eliminated so long as the proportion of air voids remains constant. A series of tests was carried out to verify the correctness of this method. Known weights of dry sand were moistened by adding known quantities of water and the moisture content was then determined by the inundation method. Fig. 3 shows the agreement between the known moisture content and that determined by the inundation method. Only the points for dry sand are considerably in error. This is attributed to the fact that the apparent specific gravity, 2.62, was

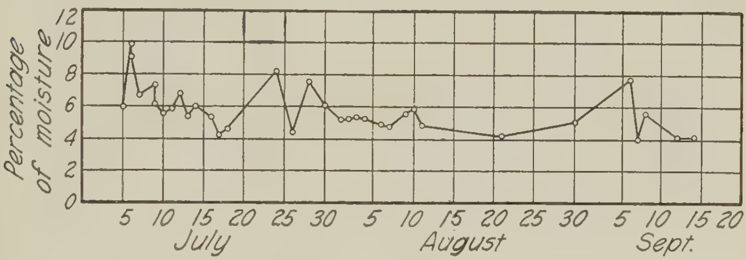


FIG. 4.—DAILY MOISTURE DETERMINATIONS FOR SAND.

obtained from inundation measurements of moist sand, whereas dry sand is likely to entrain considerably more air voids than wet sand. A difference of a little more than 1 per cent of air voids for the dry and the moist sand tests would account for the error found in Fig. 3. Apparently the method is satisfactory for more than 1 per cent of moisture. When less than 1 per cent of moisture is present it can be used by moistening the test sample after weighing and before inundating. Its advantage over the drying and weighing of the sand is the fact that with the inundation method the moisture content of the sand may be known in a few minutes after taking the sample. Fig. 4 gives the percentages of moisture for a considerable period during the summer of 1923. The sand appeared to hold about 5 per cent of moisture for a considerable length of time. The occasional increases in moisture content shown in that figure correspond in a general way to the rainy weather. Differences in position in the pile from which the sample was taken, also may help to account for the variation in moisture.

* Smith and Slater "Inundation Methods for Measurements of Sand in Making Concrete." Fig. 6, p. 231, Proc. A. C. I., v. 19.

Slump and Diameter of Slump Specimens.—Slump tests were made upon the specimens taken for compressive strength. Since, as the work progressed, little relation could be seen between the measured slump and the compressive strength, measurements of the diameter of the slump specimens were taken after the slump had been measured. The slump and

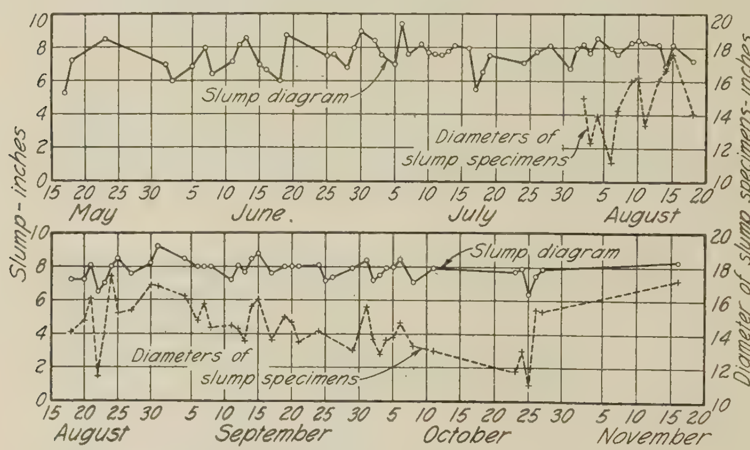


FIG. 5.—DAILY SLUMPS AND DIAMETERS OF SLUMP SPECIMENS.

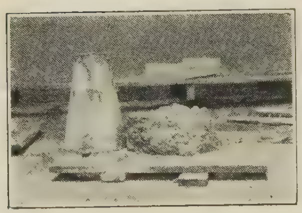


FIG. 6.—SLUMP TEST; CYLINDER 135.

FIG. 7.—SLUMP TEST; CYLINDER 252.

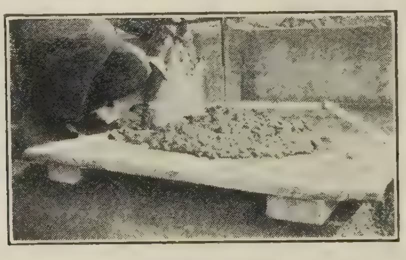


FIG. 8.—SLUMP TEST; CYLINDER 255.

diameter measurements as taken day after day are shown in Fig. 5. Figs. 6, 7, and 8 give views which show considerable range in slump. Fig. 6 shows a specimen in which there was a very large proportion of fine material in the crushed stone. Such concrete gave the appearance of fatness, but required a considerable quantity of water to give it workability. There was no direct evidence that its strength was deficient. The water-cement ratio for the specimen shown in Fig. 6 was 1.04 (excluding moisture in coarse aggregate) and the strength was 2065 lb. per sq. in. The specimen shown in Figs. 7 and 8 did not have the excess fine material in the coarse aggregate. Their water-cement ratios are not known, but were probably about 1.1, the value for other specimens taken on the same day. Their strengths were 2930 and 2065 lb. per sq. in.

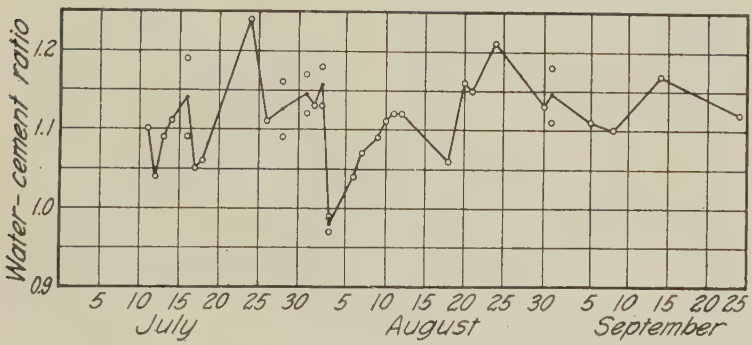


FIG. 9.—DAILY DETERMINATIONS OF WATER-CEMENT RATIO.

Water-Cement Ratio.—It was not possible to determine exactly the water-cement ratios for the field concrete. The readings of the water regulator on the mixer gave fairly accurate indication of the amount of water added, but the amount of moisture in the stone was unknown and the moisture in the sand was determined at relatively few positions in the pile. However, since a variation of 1 per cent of moisture content of the sand caused a variation of the water-content ratio of only .034, the error introduced by using the values from Fig. 4 in working out the water-cement ratio must be small. For days on which the moisture content of the sand was not determined it was taken as 5 per cent. The moisture in the crushed stone was assumed to be necessary for wetting the aggregate and satisfying the absorption requirements. It was, therefore, neglected in computing the water-content ratios given in Fig. 9 and used in Fig. 10. On some days sufficient data for a satisfactory estimate of the water-cement ratio were not available.

Strength of Concrete.—Each strength shown in Fig. 10 and 11 is the average for all the specimens of its particular kind that were made on the day indicated. On some days only two specimens were made and on

others as many as four or six of one kind. The cumulative daily averages are the averages of the daily averages up to the time in question. They do not indicate the trend of results except for the first few specimens, because after, say, 100 days the effect of one day's specimens falling as much as 50 per cent below the average up to that time would be to reduce

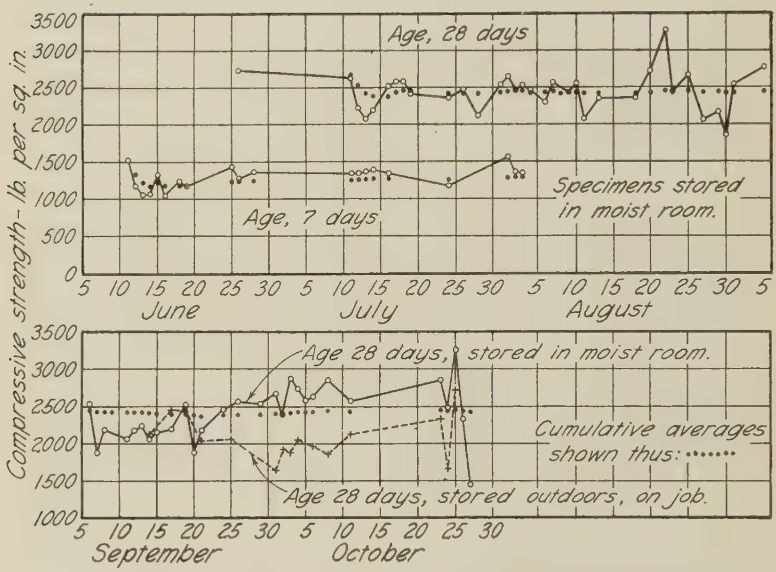


FIG. 10.—DAILY STRENGTH OF 1 : 2.5 : 3.33 CONCRETE.

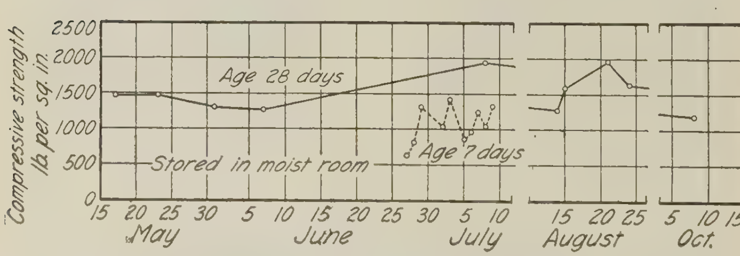


FIG. 11.—DAILY STRENGTHS OF 1 : 2 : 4 CONCRETE STORED IN MOIST ROOM.

the average by only half of 1 per cent. However, in any strength specification a permissible lower limit of average strength would probably be set and it seems worth while to show the average strength at all times.

As cool weather and the date for the first use of the stadium approached some of the specimens were left where they were made until the time for testing in order to obtain an idea of the strength of the concrete in the structure. The strengths of these specimens are shown in Fig. 10.

The average of these strengths was 15 per cent less than that for similar specimens stored in the moist room. This difference may have been due partly to too rapid drying out, but probably was due mostly to the lower storage temperature for the specimens stored out-doors.

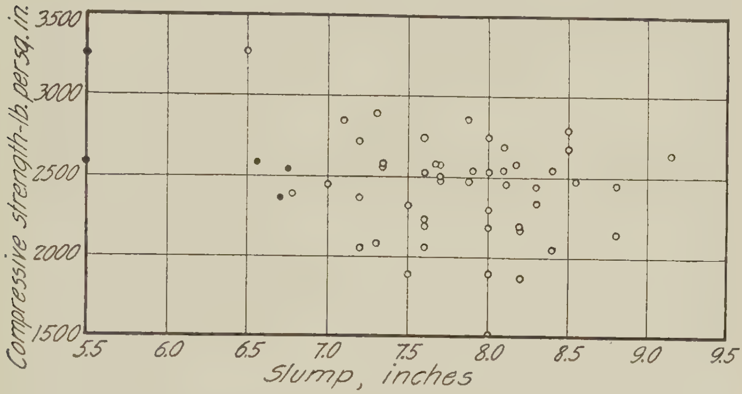


FIG. 12.—RELATION BETWEEN SLUMP AND 28-DAY STRENGTHS FOR 1: 2.5: 3.33 CONCRETE STORED IN MOIST ROOM.

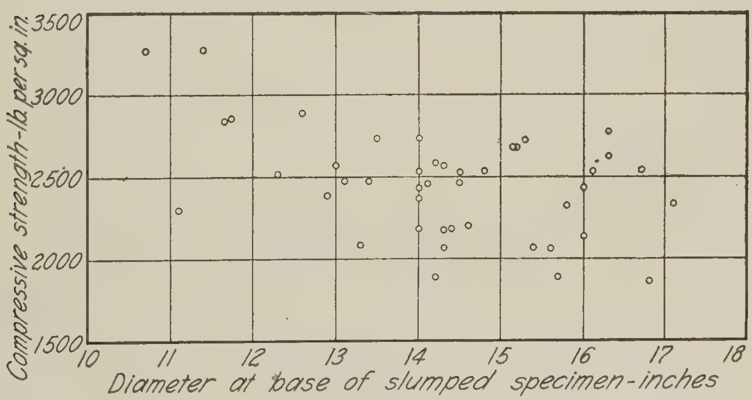


FIG. 13.—RELATION BETWEEN DIAMETER OF SLUMP SPECIMENS AND STRENGTH.

The strengths as ordinates with slumps as abscissas are given in Fig. 12 for 1: 2.5: 3.33 concrete. There is a slight indication of greater strength with low than with high slump, but this impression is given by about four high strengths while all the rest indicate no relation at all. For the points shown as solid circles the corresponding diameters were not measured. With this exception the slumps in Fig. 12 and the diameters in Fig. 13 were measured on the same specimens. Fig. 13 indicates that the strength

was somewhat more distinctly related to the diameter than to the slump of the specimen.

Fig. 14 gives the relation between the water-cement ratio and the strength of 1: 2.5: 3.33 concrete. The points plotted are generally the average of from 2 to 4 specimens taken on the same day.

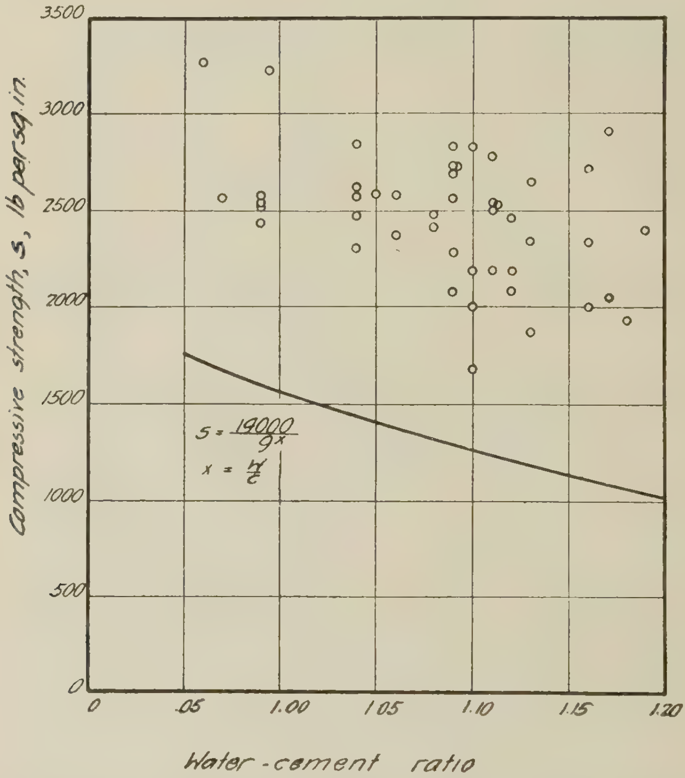


FIG. 14.—RELATION BETWEEN WATER-CEMENT RATIO AND 28-DAY COMPRESSIVE STRENGTH OF 1: 2.5: 3.33 CONCRETE STORED IN MOIST ROOM.

There is danger that this diagram may be misleading since the water-cement ratio appears as the only variable affecting the strength. The variations in water-cement ratio are largely accidental, and possibly not much greater proportionally than the variations in the proportions of cement, sand and stone.

In Fig. 15 and 16 are given the compressive strength curves for the 1: 2.5: 3.33 and the 1: 2: 4 concretes. That is, the strengths for individual cylinders are arranged in the order of their magnitude. These curves give

TABLE 4.—AVERAGE STRENGTH AND UNIFORMITY OF STRENGTH OF CONCRETE.

Mix.	Age, days.	Storage.	Average Strength, lb. per sq. in.	Percentage of All Specimens.			Range of Strength, per cent of Average.
				Below 80 per cent of Average.	Above 120 per cent of Average.	Between 80 per cent and 120 per cent of Average.	
1:2:4	7	Moist room	1100	25	20	55	55–140
1:2:4	28	Moist room	1550	12	17	71	40–140
1:2.5:3.33	7	Moist room	1350	11	14	75	40–140
1:2.5:3.33	28	Moist room	2400	11	12	77	55–135
1:2.5:3.33	28	Job	2050	10	12	78	65–145

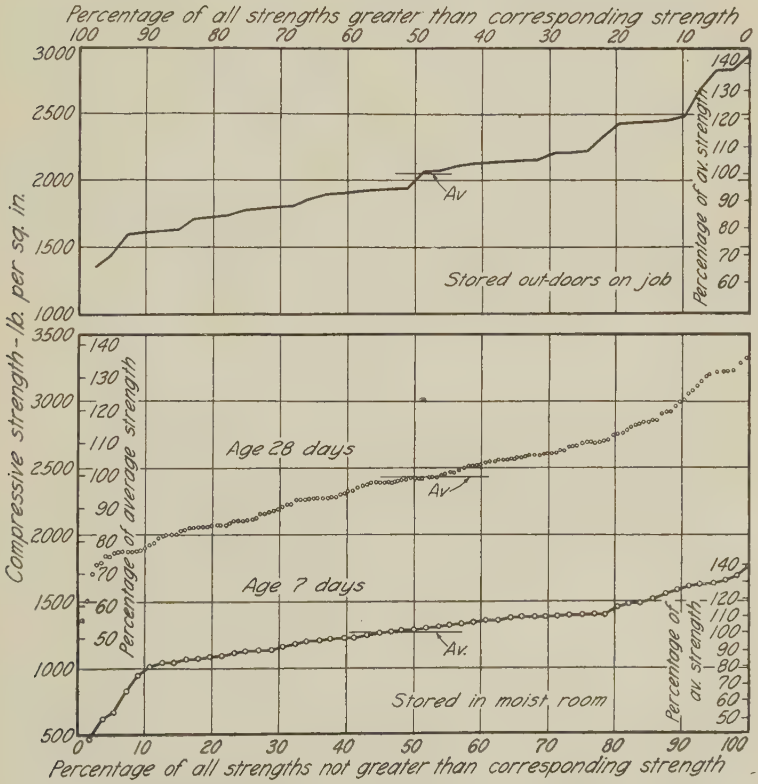


FIG. 15.—PROGRESSIVE STRENGTH CURVE FOR 1 : 2.5 : 3.33 CONCRETE.

some information on the uniformity of the strength. The values given in Table 4 taken from Fig. 15 and 16 show that for the 1 : 2.5 : 3.33 concrete a larger proportion of the strengths lay between 80 and 120 per cent of the average than for the 1 : 2 : 4 concrete. The specimens stored on the job showed up as well for uniformity of strength as those stored in the moist room.

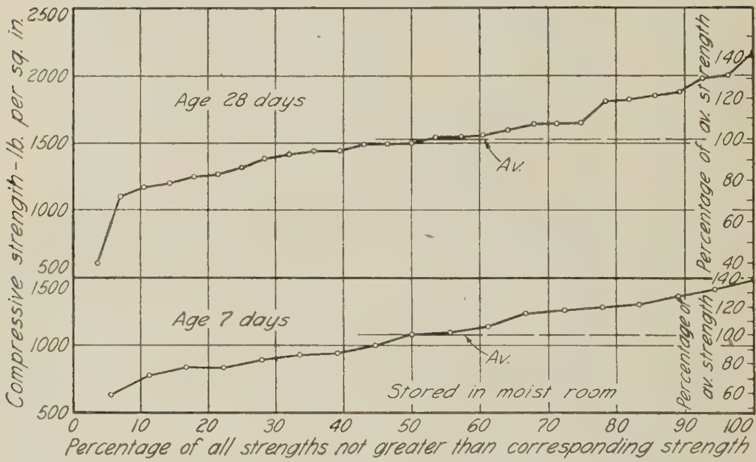


FIG. 16.—PROGRESSIVE STRENGTH CURVE FOR 1 : 2 : 4 CONRETE STORED IN MOIST ROOM.

SUMMARY.

1. The water measuring device on the mixers used were found to show quite accurately the amount of water added to the batch.

2. The inundation method of determining moisture in the sand was found to be quite reliable. It is believed that this method can be adapted also to the measurement of moisture in the coarse aggregate. The method has the advantage of giving the information needed regarding the moisture content so quickly that it may be used in determining the amount of water to be added in the mixer.

3. Laboratory and field studies showed that a concrete having considerably more sand than that necessary to give the maximum density of the concrete was required in order to secure a sufficient degree of workability.

4. It was difficult to see much relation between the strengths and the measured slumps. The relation between the strengths and the diameters of the slump specimens was only slightly more distinct. All the errors

involved in manipulation and observation are included in the slump, diameter, and strengths recorded and a less consistent relation should be expected than in cases where the slump is deliberately varied.

5. The water-cement ratio determined on the basis of the moisture in the sand plus the water added in the mixer and neglecting the moisture in the stone varied between 0.97 and 1.24; the average was approximately 1.12.

6. The average strength of the 1:2.5:3.33 concrete was somewhat higher than might have been expected on the basis of the average water-cement ratio. The strength at seven days for this concrete was approximately one-half that at twenty-eight days.

The average strength of the 1:2:4 concrete was slightly less than might be expected from the average water-cement ratio. Its strength at seven days was approximately two-thirds of that at twenty-eight days.

The specimens stored on the job gave an average strength about 15 per cent lower than that of the same kind of concrete stored in a moist room. Most of the field stored test specimens were made in September and October.

7. The indication from these tests is that it is entirely feasible with ordinary methods of concreting to secure concrete of a strength not less than that aimed at in design.

APPENDIX: INUNDATION METHOD OF MAKING MOISTURE DETERMINATIONS.

The equation which forms the basis of moisture determination by the inundation method is:

$$W = W_s + V - A - \frac{W_s}{g} \text{ where} \qquad (1)$$

W = total weight in grams.

W_s = weight in grams of aggregate exclusive of contained moisture.

V = volume of container in cubic centimeters.

A = volume of air voids in cubic centimeters.

g = specific gravity of aggregate.

From eq. 1.

$$W_s = W - \frac{(V\text{-}A)}{1\text{-}1/g} = \frac{W\text{-}V\ (1\text{-}r)}{1\text{-}1/g} \qquad (2)$$

Where r is the ratio of volume of air voids to total volume V.

Let m = ratio of weight of moisture in aggregate to weight of aggregate exclusive of moisture.

Let W_m = weight of aggregate plus the inundated moisture, that is, the original weight of the moist sample.

From (2)

$$\frac{W_m}{W_s} = \frac{(1\text{-}1/g)\,W_m}{W\text{-}V\,(1\text{-}r)} \qquad (3)$$

$$m = \frac{W_m}{W_s} - 1 = \frac{(1\text{-}1/g)\,W_m}{W\text{-}V\,(1\text{-}r)} - 1 \qquad (4)$$

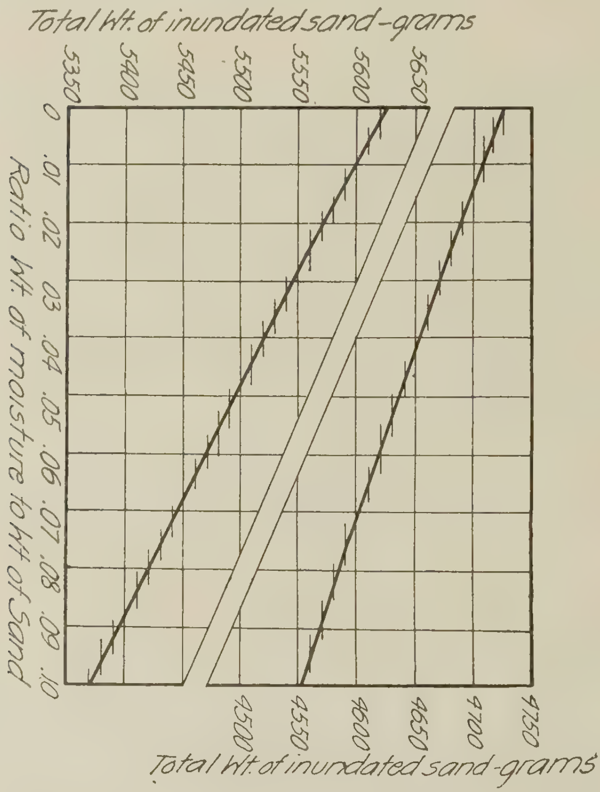


FIG. 17.—DIAGRAM FOR DETERMINATION OF MOISTURE RATIO IN SAND AND STONE; WEIGHT OF SAND SAMPLE 4536 G. (10 LB.); WEIGHT OF STONE SAMPLE 3000 G. (661 LB.).

It will be seen from this equation than an exact determination of the moisture ratio requires a knowledge of the percentage of air voids. Experience indicates, however, that the error due to neglecting the air voids can be practically eliminated by using for g the apparent specific gravity of the aggregate with its entrained air. This is done by using equation (4) for the determination of g where in W_m, W and V are known and r is neglected. If the percentage of air voids were always the same this method

would be exact, and only when there are considerable variations in the percentage of air voids will an appreciable error be introduced. This is illustrated in Fig. 17. The data of that figure were obtained by using for g the average apparent specific gravity from equation (4) when the moisture ratios were 0.0167, 0.0334, and 0.0667. When dry sand was tested it seems that enough air voids were introduced to give an apparent moisture content of about 1 per cent.

The inundation method of determining moisture content of the sand has been used regularly on the stadium job. It is now being used for the stone, but has not been very fully verified. In operation it is possible to simplify the work greatly by using a fixed weight W_m, of the sample of moist aggregate.

For the determinations a container having a capacity of 2824 cc. (.0997 cu. ft.) and weighing 5130 g. is used. A 4536 g. (10 lb.) sample of moist sand and a 300 g. (6.61 lb.) sample of crushed stone is used. The specific gravity of the sand was found to be 2.62 and that of the crushed stone 2.73. The curves given in Fig. 17 are graphs of equation (4) with the proper values for g and W_m inserted. In actual operation the weight of the container and the weight of a plate-glass cover are added to the ordinates in order to avoid the necessity of computing the net weight for each determination. The use of the plate-glass cover provides a convenient means to insure the filling of the container exactly full and to avoid the spilling of water in handling the full container. The weights of the original sample and of the container full of sand and water are the only experimental data needed for a moisture determination. By using samples of standardized weight the moisture may be read from a diagram such as that given in Fig. 17 with accuracy sufficient for practical work.

FIELD TESTS OF CONCRETE USED ON CONSTRUCTION WORK.

By W. A. Slater*, Director of Tests

and

Stanton Walker†, Resident Testing Engineer.

Introduction.—The following is an abstract of a report which is under preparation on two series of tests of "field" concrete.

The tests were undertaken for the purpose of establishing a means for introducing into practice some of the improvements in the technique of concrete-making which recent researches have indicated to be desirable. In planning the investigations a program of tests was first outlined by the Joint Committee on Standard Specifications for Concrete and Reinforced-concrete and, later, modifications were made as a result of conferences with the General Committee of Contractors. As stated in the program thus agreed upon, the immediate purpose of the tests was to test "the practicability of the recommendations of the Joint Committee on Standard Specifications for Concrete and Reinforced-concrete for securing the compressive strength desired in concrete produced under field conditions."

The organizations which participated in both the planning and execution of the tests were:

Joint Committee on Standard Specifications for Concrete
and Reinforced-concrete,
Bureau of Standards,
Portland Cement Association.

The principal expenses of the investigations were borne by the General Committee of Contractors and the Portland Cement Association. For valuable co-operation in carrying out the tests, acknowledgment is made to the following organizations and to members of their staffs who participated in the work:

(1) Stone & Webster, Inc.,
(2) Municipal Testing Laboratory of Philadelphia,
(3) Pennsylvania State Highway Department,
(4) Victor Talking Machine Co.,
(5) Central Railroad of New Jersey.
(6) Henry Steers, Inc.,
(7) Columbia University.

Site and Plant.—The first series of tests was carried out during the erection of an eight-story, reinforced building at Camden, N. J., for the Victor Talking Machine Co. This building is a flat slab structure about

---

*Engineer-Physicist, Bureau of Standards.

†Associate Engineer, Structural Materials Research Laboratory.

425 ft. in length, from 70 to 90 ft. in width and 8 stories in height. The construction involved the placing of about 12,000 cu. yd. of concrete. There were two concrete plants; an east and a west plant. Practically all the specimens were taken from the east plant. Stone & Webster, Inc., were the engineers and contractors. The concrete was raised by means of hoisting towers to the working level and dumped into hoppers from which it was wheeled in buggies to the place of deposit.

The second series of tests was carried out during the construction of piers for a bridge on the Central Railroad of New Jersey across Newark Bay between Bayonne and Elizabeth, N. J. Henry Steers, Inc., was the contractor. At Newark Bay the concrete from which the test specimens were taken went into the piers. The piers are of two sizes, 43 and 47 ft.

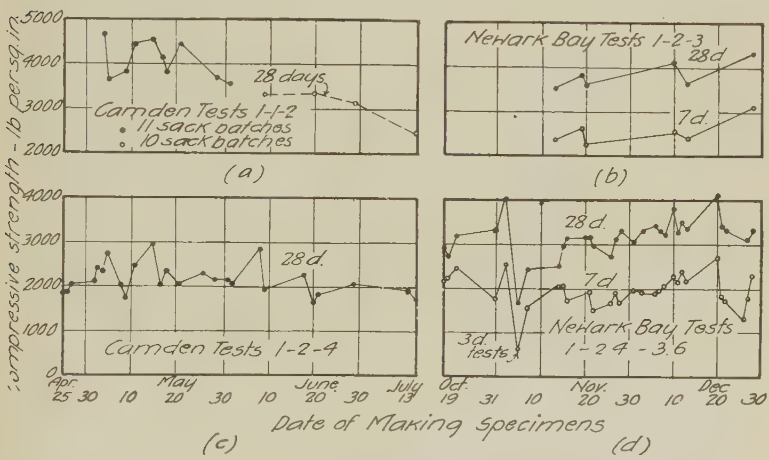


FIG. 1.—DAILY AVERAGE COMPRESSIVE STRENGTH.

long respectively, and both are 22½ ft. wide at the bottom. The height was about 50 to 55 ft., and each pier, including the caisson bottom, used about 750 cu. yd. of concrete. The concrete was distributed by means of a tower, a system of chutes and a vertical spout. On account of the fact that the piers are in sea-water, concrete of a high grade was necessary.

Outline of Tests.—The Camden tests were divided into six series, each with a specific purpose in view. The following gives a brief summary of each series as outlined, together with comments concerning variations from the outlined procedure. The program for the Newark Bay tests was essentially similar to that for the Camden tests.

Series 1 had for its purpose the determination of the range of consistency to be expected throughout the work as determined by slump test. Hourly slump and flow tests were made throughout the progress of the job for the concrete produced by the east mixer plant.

Series 2 was designed to show the range in strength of concrete to be expected throughout the job. Three 6 x 12-in. concrete cylinders were made for each hour of concreting from the east mixer plant, for test at 28 days. For about 25 per cent of the batches sampled 6 specimens were made from each batch to be tested one at 7 days, one at 3 months, one at 1 year, and 3 at 28 days.

Series 3 was designed to show the relation of strength of field specimens to the strength of the concrete in the structure. For this purpose

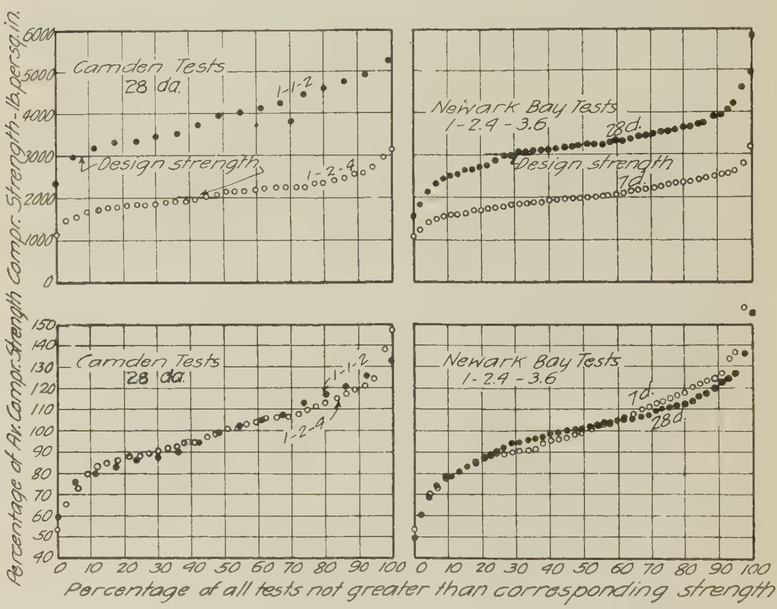


FIG. 2.—PROGRESSIVE STRENGTH CURVES FOR TEST CYLINDERS.

nine concrete slabs were cast from each of which nine cores 4½ in. in diameter were cut at about 28 days to be tested, 3 at 28 days, 3 at 3 months, and 3 at 1 year.

Two test columns each 10 ft. long were made; one was 12 in. in diameter and one was 12 in. square. Four cylinders were sawed from the round column and four prisms from the square column for test at 28 days.

In *Series 4* batches were mixed to show the variation in concrete strength for different proportions of sand and pebbles. These batches used aggregates in which the sand was 50, 40, 35, 30 and 25 per cent respectively. Parallel tests were made using small batches mixed by hand in the field laboratory.

Series 5 was a comparison of the strength of field-made test specimens with that of similar specimens made in the Laboratory. In general,

batches were made in the field for 4 and 8-in. slumps for 1: 3: 6, 1: 2: 4, 1: 1½: 3 and 1: 1: 2 mixes. Samples of the job materials were shipped to the U. S. Bureau of Standards, for the purpose of duplicating these proportions and slumps in the Laboratory.

Series 6 is omitted from this abstract.

Series 7 was carried out to furnish a comparison of the concrete-making qualities of the different lots of the cements used on the job. Samples taken weekly, were shipped to the Bureau of Standards for standard cement tests and for test in 1: 2: 4 concrete at 7 days, 28 days, 3 months and 1 year, using a standardized mixture of fine and coarse aggregates.

Materials and Proportions.—For the Camden job the concrete was mixed in batches of approximately 1 cu. yd. The mixes were nominally 1: 2: 4 and 1: 1: 2. For these mixes respectively 6 and 11 bags of cement

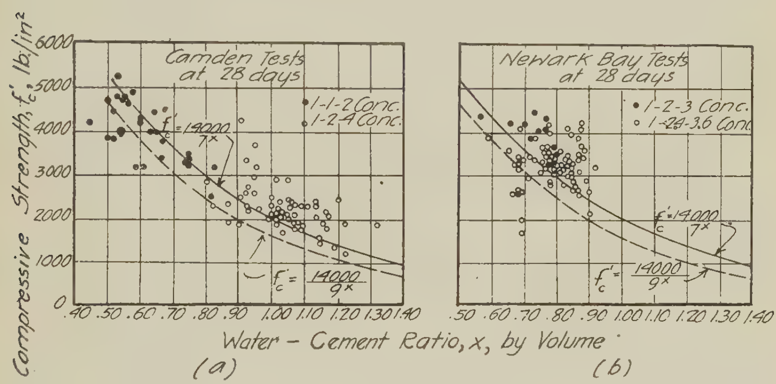


FIG. 3.—COMPRESSIVE STRENGTH PLOTTED AGAINST WATER CEMENT RATIO.

per batch were used. Identical quantities of aggregate were used in the two classes of concrete. Two brands of portland cement, Pennsylvania and Phoenix, were used. For most of the work Pennsylvania cement was used in the east plant. The aggregates were washed sand and washed pebbles from the plant of the De Frain Sand and Gravel Co.

For the Newark Bay job the concrete was mixed in batches of about 1 cu. yd. in a mixer located on a large scow. The nominal mixes were 1-2.4-3.6 and 1-2-3 and 5 and 6 bags of cement per batch respectively, were used. The aggregates were washed sand and washed pebbles from the plant of Henry Steers, Inc., near Northport, Long Island.

General Procedure for Testing.—The test methods generally were in accordance with recommendations of the American Society for Testing Materials.

Sampling Concrete.—At Camden samples of concrete were taken by filling simultaneously with the filling of the buggy, a sheet-iron box placed in a buggy and extending its full length. The final sample consisted of a mixture of two such samples from each batch.

At Newark Bay the concrete was shoveled from the hoisting bucket after it had received half of the batch from the mixer. To facilitate the sampling the flow from the mixer was stopped during the taking of the sample. Sufficient concrete was shoveled for 6 or 12 cylinders.

Measurement of Workability.—As a measurement of workability of the concrete both the slump and the flow tests were used. The slump test was made in accordance with the tentative specifications D62-20T*, of the American Society for Testing Materials adopted by the Joint Committee as a standard.

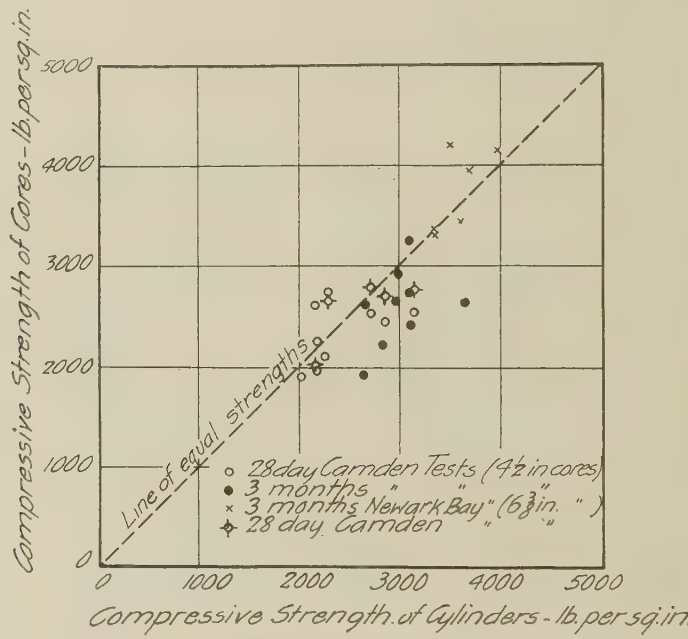


FIG. 4.—COMPARISON OF COMPRESSIVE STRENGTH OF CORES AND CYLINDERS.

The flow test was made using a flow table furnished by the U. S. Bureau of Public Roads.† The increase in the base diameter, expressed as a percentage of the original diameter is reported as the flow.

The 6 x 12-in. test cylinders were made near the place of deposit of the concrete and were stored temporarily in damp sand in the field laboratory. After a period varying from four days to three weeks they were removed

*A later tentative specification, D-138-20T provides that the slump test shall be made 3 minutes after molding the specimen. The former method was used and seems preferable.

†For details of this flow-table see "Inundation Methods for Measurement of Sand in Making Concrete," by G. A. Smith and W. A. Slater, Proc. A. C. I., vol. xix, (1923) p. 227.

Table 1.—Tests of 1:2:4 Concrete at Camden.

Compression tests of 6 x 12-in. cylinders made in field.
Aggregate: Sand and pebbles from De Frain Sand and Gravel Co., Philadelphia.
Age at test, 28 days.
Samples from 6-sack batch of about 1 cu. yd.
Cylinders removed from molds and stored in damp sand indoors after 24 hours.
Specimens cured in damp sand; tested damp.

Batch No.	Date Made (1923)	Water Ratio	Void Cement Ratio	Slump, in.	Flow, per cent	Weight of Concrete, lb. per cu. ft.	Compressive Strength, lb. per sq. in.				Mean Variation per cent
							(1)	(2)	(3)	Average	
1a....	4–25	....	....	8.5	..	148	2040	2230	2120	2130	3.1
2a	4-25	....	....	8 6	..	147	1700	1470	1530	1570	5.7
3 ...	4–26	....	....	8 0	..	144	1690	1850	1870	1800	4.2
4....	4–26	....	....	8.5	..	143	2120	1800	2080	2000	6.6
5.....	4–26	....	....	8.0	..	146	1920	1880	1780	1860	2.8
6a....	4–27	....	....	7.0	..	144	2110	1960	2090	2050	3.7
7a....	5–2	0.87	0.96	8.5	90	146	2420	2260	2270	2320	3.0
8a....	5–2	0.87	0.86	9.5	100	149	1950	1900	1810	1890	2.5
9.....	5–3	0.92	0.97	10.0	130	147	2260	2190	2340	2260	1.3
11b....	5.3	0.92	0.91	8.0	80	148	2490	2560	2540	2530	1.0
15b, c..	5–4	0.80	0.88	7.5	80	148	2780	2830	2850	2820	0.9
18b....	5–4	0.82	0.86	5.0	65	149	2290	2220	2270	2260	1.1
19b...	5-4	0 90	0 96	7.0	95	147	1770	1930	1820	1840	3.2
21d...	5-5	0 92	0 99	9.5	120	146	3150	3440	3320	3300	3.1
22e....	5–5	0.93	0.95	8 5	100	147	1980	2240	2240	2150	5.6
23b....	5–8	0.96	0.91	7.5	70	149	2230	2230	2150	2200	1.6
24b....	5–8	1.01	1.01	8.0	70	146	1860	1850	1880	1860	0.5
25b....	5–9	1.08	1.01	8.5	90	147	1630	1770	1920	1770	4.3
27b....	5–11	0.95	0.91	8.8	100	149	3200	3080	3060	3110	1.9
29f....	5–11	0.91	....	7.0	55	150	4300	4080	4220	4200	2.1
30b....	5–11	0.91	0.94	7.5	70	148	2420	2430	2470	2440	0.8
31.....	5–15	1.01	0.90	6.0	20	151	3090	2900	2620	2870	5.7
32.....	5–15	0.93	0.96	7.5	80	148	2780	2720	2610	2700	2.3
34c....	5–15	0.95	0.98	6.5	50	147	3600	3760	3580	3650	2.1
35.....	5–15	0.94	1.05	6.5	30	145	2560	2610	2480	2550	2.0
36.....	5–15	0.94	1.08	7.0	40	144	2860	2970	2910	2910	1.3
41.....	5–17	1.00	1.15	8.8	115	143	2180	1960	2040	2060	3.9
42.....	5–18	1.00	1.13	7.5	105	144	2430	2360	2530	2440	2.6
43.....	5–18	1.00	0.93	8.0	105	149	2480	2120	2170	2260	5.1
44.....	5–18	1.20	1.01	5.8	50	150	2420	2530	2320	2420	2.9
47.....	5–21	1.13	1.08	7.5	85	147	2250	2300	2550	2370	5.2
48.....	5–21	1.08	1.13	6.8	70	145	1760	1730	1800	1760	1.3
51.....	5–26	1.07	1.03	6.8	..	148	2220	2370	2280	2290	2.3
54.....	5–29	1.02	1.01	7.3	90	147	2040	2000	2140	2060	2.6
55.....	5–29	1.13	0.98	8.0	100	149	2220	2210	2240	2220	0.5
57.....	6–1	1.04	1.06	7.5	90	146	1900	1910	1980	1930	1.7
59.....	6–1	1.04	1.06	6.2	65	145	2320	2520	2290	2380	4.1
60.....	6–1	1.02	1.06	6.8	70	146	2130	2240	2130	2180	1.7
61g....	6–1	1.05	1.05	6.2	75	146	2020	2100	2150	2090	2.2
62.....	6–2	1.15	1.09	7.3	78	147	1780	1920	1850	1850	2.5
63.....	6–2	1.10	1.02	4.0	40	148	2110	2450	2290	2280	5.1
73h....	6–8	1.05	1.11	8.8	95	144	2920	2700	3020	2880	4.2
74.....	6–9	1.17	1.16	7.5	85	144	2180	1990	2140	2100	3.7
75.....	6–9	1.22	1.22	7.0	70	143	1730	1970	1870	1860	4.5
78.....	6–9	1.08	1.03	6.0	70	146	1910	1880	1830	1870	1.6
83i....	6–18	1.05	1.01	7.8	105	145	2410	2600	2260	2420	4.8
84.....	6–18	1.02	1.01	7.2	105	146	2380	2310	2450	2380	2.0
85.....	6–18	1.01	0.91	6.8	80	150	2160	2180	2230	2190	1.2
86.....	6–18	0.99	0.98	8.0	110	148	1930	2060	1940	1980	2.9
87.....	6–18	1.11	1.07	6.5	90	146	2190	2310	2370	2290	2.9
90j....	6–20	1.22	1.05	9.5	150	146	1190	1140	....	1160	2.2
92.....	6–20	1.08	1.05	7.8	108	147	1910	1830	1670	1600	5.0
94.....	6–20	1.07	1.04	8.0	110	148	1870	2140	2080	2030	5.3
95.....	6–20	1.03	1.05	8.5	130	146	1700	1650	1660	1670	1.2
97k....	6–21	1.07	1.16	8.0	125	144	1830	1850	1760	1820	1.5
101.....	6–29	1.17	1.20	8.0	120	143	1970	2140	2100	2070	3.2
112.....	7–11	1.01	1.06	6.0	38	146	1960	2010	1770	1910	5.1
121.....	7–13	1.17	1.11	5.7	70	147	1660	1780	1770	1740	2.9
122.....	7–13	1.14	0.98	8.2	105	150	1510	1430	1640	1530	5.0
123.....	7–13	1.14	1.07	7.7	100	147	1520	1470	1440	1480	2.0
124.....	7–13	1.19	....	7.5	93	...	1350	1320	....	1340	1.1
125.....	7–13	1.32	1.14	7.7	90	147	1940	1840	1860	1880	2.1
127.....	7–13	1.05	1.03	6.5	60	147	2230	2220	2340	2260	2.5
Average.	....	1.03	1.02	7.5	85	147	....	....	....	2190	2.9

a 1 bag of hydrated lime to 6 bags of cement.
b Additional sand and pebbles added to measured quantity.
c Probably first 1:2:4 batch after a run of 1:1:2 concrete.
d First 1:2:4 batch following a run of 1:1:2 concrete.
e Additional sand.
f Sweepings from platform added.
g Slightly more than average amount of sand.
h Usual batch.
i 7-bag batch.
j Excess of sand, dirty aggregate, scrapings from dock.
k More than average amount of sand; concrete appeared dirty.
l Duplicate specimens made of Stone and Webster.

to the Testing Laboratory and again stored in damp sand, where they remained until they were tested.

Tests of Aggregate.—In general the samples of sand and gravel for moisture determinations and sieve analyses were taken as the materials were discharged from the overhead bins into the mixer hopper. The sieves

TABLE 2.—TESTS OF 1:1:2 CONCRETE AT CAMDEN.

Compression tests of 6 x 12-in. cylinders made in field.
Specimens cured in damp sand until test; tested damp.
Age at test, 28 days.
Samples from 11-sack batch of about 1 cu. yd.

Batch No.	Date Made (1923)	Water Ratio	Void Cement Ratio	Slump, in.	Flow, per cent	Weight of Concrete, lb. per cu. ft.	Compressive Strength, lb. per sq. in.				Mean Variation, per cent
							(1)	(2)	(3)	Average	
10*a*	5–3	0.54	0.52	9.0	80	146	4610	3540	3710	3950	10.7
12*b*	5–4	0.57	0.79	9.0	100	149	4820	4220	4749	4590	5.4
13*b*	5–4	0.53	0.50	8.0	85	151	5190	5240	5200	5210	0.4
14*b*	5–4	0.54	0.60	9.0	85	145	4980	4770	4330	4690	5.2
15*b*	5–4	0.58	0.53	8.0	100	151	4900	4700	5060	4890	2.5
17*b*	5–4	0.51	0.57	7.5	85	151	3610	3940	3840	3800	3.3
20*d*	5–5	0.60	0.61	9.0	90	147	3520	3740	3610	3620	2.1
26	5–9	0.60	0.62	10.0	110	147	4030	3500	4050	3860	6.2
28*e*	5–11	0.52	0.60	8.0	90	151	4420	4450	4450	4450	0.3
33	5–15	0.55	0.53	8.0	68	149	4940	4650	4680	4760	2.6
37*f*	....	0.45	0.53	7.5	50	145	4250	4090	4160	4170	1.4
38*g*	5–15	0.50	0.61	8.5	100	147	4840	4510	4760	4700	2.8
39	5–17	0.60	0.69	9.5	110	145	3960	4190	4280	4140	3.0
45	5–18	0.67	0.65	7.8	90	147	3520	3730	3940	3730	3.5
46*c*	5.18	0.64	0.58	8.0	105	151	4000	3740	4080	3940	3.4
49*b*, *h*	5–21	0.65	0.65	9.0	110	147	4370	4500	4460	4440	1.1
52	5–29	0.60	0.60	8.5	90	149	4250	4080	4460	4260	3 1
53	5–29	0.50	0.60	8.5	138	149	2980	3120	3360	3150	4.4
56	6–1	0.61	0.69	8.0	85	144	3310	3100	3040	3150	3.4
58	6–1	0.63	0.68	7.5	75	145	3920	3940	4100	3990	1.9
76*i*	6–9	0.74	0.76	7.0	80	146	3390	3230	3160	3260	2.6
79*j*	6–9	0.75	0.65	9.5	130	147	3460	3300	3560	3440	2.7
88*i*	6–9	0.74	0.69	8.5	115	149	3200	3340	....	3270	2.1
91*j*	6–20	0.75	0.63	6.8	65	148	3300	3260	3380	3310	1.3
93	6–20	0.66	0.65	7.5	65	147	3360	3400	3410	3390	0.6
106*i*, *k*	6–29	0.82	0.77	9.0	120	146	3180	3290	2960	3140	3.9
126*i*	7–13	0.81	0.76	9.0	108	147	2640	2360	2440	2480	4.3
Average	....	0.62	0.63	8.4	94	148	....	....	....	3910	3.1

a Small amount of sand, not measured, added to measured quantity.
b Additional sand and pebbles added to measured quantity.
c 12 sacks of cement in batch.
d Additional sand.
e Additional sand and pebbles.
f 14 sacks of cement in batch.
g About half of usual amount of sand and pebbles.
h 29 days old.
i 10 sacks of cement in batch.
j 9 sacks of cement in batch.
k Special batch for effect of quantity of cement

specified by the American Society for Testing Materials were used in making the sieve analyses (C41-22).

The moisture content was found by determining the loss of weight on drying of the sample used for sieve analyses.

Uniformity of Strengths.—The strengths on which the study of uniformity is based are given in Tables 1 and 2 for the Camden tests and in

TABLE 3.—COMPRESSION TESTS OF 1:2.4:3.6 CONCRETE AT NEWARK BAY.

Compression tests of 6 by 12-in. cylinders made in field.
Aggregate: Sand and pebbles from near Northport, Long Island; furnished by Henry Steers, Inc. Portland cement was used.
Mix, 1:6 damp and loose, separated volumes (5-bag batches).
Cylinders removed from molds and stored in damp sand indoors after about 24 hours.
Samples selected at mixer from 1 cu. yd. batch.
Specimens cured in damp sand until test; tested damp.

Batch No.	Date Made (1923)	Water Ratio	Void Cement Ratio	Slump, in.	Flow, per cent	Compressive Strength, lb. per sq. in.								Mean Variation of Strength, per cent	
						7 Days				28 Days					
						(1)	(2)	(3)	Aver.	(1)	(2)	(3)	Aver.	7 Days	28 Days
1	10–19	....	....	1.5	25	1950	1820	1850	1870	3045	2490	2170	2570	2.7	12.4
2	10–19	....	....	3.2	40	2260	2580	1900	2250	3035	3030	3135	3065	10.2	1.5
3	10–19	....	....	3.0	25	2320	2030	2280	2210	3090	2875	3150	3040	5.4	3.6
4	10–19	0.75	0.84	3.2	40	2240	2250	2100	2200	3165	3015	3070	3085	2.9	1.8
5	10–20	0.79	0.80	5.0	30	2390	2480	2560	2480	3160	2730	3090	2990	2.3	5.9
6*	10–20	....	....	3.0	20	2000	1950	1880	1940	2540	2480	2450	2460	2.2	1.5
7	10–22	0.59	0.72	3.8	40	2510	2730	2900	2710	4000	3970	3500	3820	5.1	5.7
8	10–22	0.66	0.66	5.5	50	1440	2020	2300	1920	2270	2720	2720	2570	16.7	7.8
9	10–22	0.66	0.78	2.0	10	2420	3180	2870	2820	3160	3520	2910	3200	9.6	6.8
10	10–31	0.69	0.80	1.8	20	1940	1760	1860	1850	4020	3430	2680	3380	3.4	13.7
11	10–31	0.69	0.80	2.0	20	1680	1530	1990	1730	2990	3240	3520	3250	9.8	5.5
12	11–2	0.69	0.76	0.0	05	2440	2590	2520	2520	4420	4020	3700	4050	2.0	6.2
13	11–5	0.69	0.84	3.0	20	730	630	600	650	1780	1640	1580	1670	7.7	4.6
14	11–7	0.69	0.76	2.8	20	1145	1100	1685	1310	2100	2040	1840	1990	19.1	5.2
15	11–7	0.69	0.84	2.0	20	1585	2120	1195	1630	2360	3080	2410	2620	19.8	11.8
16	11–7	0.69	0.72	1.5	15	1700	1930	1885	1835	2680	2500	2660	2610	5.1	3.0
19	11–14	0.90	0.84	2.5	17	1670	1820	1820	1770	2520	2600	2580	2570	3.8	1.2
20	11–14	0.90	0.96	5.0	25	1780	2120	1950	1950	2200	2100	1990	2100	5.8	3.3
21	11–14	0.82	0.84	1.8	17	2510	2560	2530	2530	2720	3100	2460	2760	0.7	8.2
22	11–14	0.87	0.86	6.0	60	2110	1820	1780	1900	2620	2560	2600	2590	7.2	0.9
23	11–15	0.89	0.90	...	20	1810	2260	2060	2040	3000	3010	2640	2880	7.7	5.7
24	11–15	0.87	0.80	3.0	27	2260	2350	2010	2210	3260	3420	3270	3320	5.9	2.1
25	11–15	0.87	0.88	6.0	40	1930	1980	1760	1890	2690	2960	2360	2670	4.6	7.8
26	11–16	0.82	0.94	1.5	15	1880	1680	1660	1740	2900	3150	3380	3140	5.3	5.2
27	11–16	0.83	0.84	4.5	17	1560	1560	1490	1540	3280	3220	3100	3200	2.0	2.1
28	11–16	0.83	0.88	4.5	20	2060	1880	1710	1880	3150	2790	2990	2980	6.2	4.3
32	11–19	0.78	0.90	3	15	....	....	....	....	....	....	....	....	...	...
33	11–19	0.78	0.84	2	17.5	....	....	....	....	....	....	....	....	...	...
34	11–19	0.78	0.72	2.8	15	....	....	....	....	....	....	....	....	...	...
37	11–20	....	....	6	30	....	....	....	....	3130	3460	2700	3100	....	8.5
38	11–20	0.82	0.70	7	35	....	....	....	....	3140	3370	3160	3220	....	3.0
39	11–20	....	....	5.5	10	....	....	....	....	....	....	....	....	...	...
40	11–21	0.80	0.82	2.5	17.5	1920	1830	1970	1910	2340	3580	3630	3180	2.6	17.7
41	11–22	0.80	0.86	4.5	37.5	1720	1600	1480	1600	2860	3260	3100	3070	5.0	4.7
42	11–22	0.85	0.88	7	40	1540	1550	1400	1500	3270	2510	2800	2860	4.2	9.5
43	11–22	0.84	0.94	7	45	1430	1580	1490	1500	2980	3070	2990	3010	3.5	1.2
44	11–26	0.86	0.78	5.5	20	1790	1660	1610	1690	2560	2880	2720	2720	4.1	3.9
45	11–27	0.69	....	3.2	25	1990	2200	2120	2100	3460	3090	3420	3320	3.7	4.7
46	11–27	0.71	0.80	3	25	1710	1770	1650	1710	2480	3050	2760	2930	2.3	3.9
47	11–28	0.73	0.76	3.5	15	1640	1740	1760	1710	3080	3680	3320	3360	2.9	6.3
48	11–28	0.79	0.84	7.0	60	1870	1580	1590	1680	3580	3290	3070	3310	7.6	5.4
49	12–1	0.80	0.88	4	27.5	1960	2080	1960	2000	3480	2630	3140	3080	2.6	9.8
50	12–3	0.78	0.84	3	20	2170	2180	2040	2130	3420	3600	3210	3410	2.8	3.9
51	12–3	0.72	0.82	3.7	17.5	1920	1900	1920	1910	3550	3070	3690	3440	0.5	7.1
52	12–3	0.77	0.80	4	25	1910	1940	1750	1870	3180	3200	2950	3110	4.1	3.4
53	12–6	0.73	0.76	1.5	15	2120	2380	2080	2190	3650	3950	3840	3810	5.6	2.9
54	12–6	0.80	0.88	5	30	1770	1840	1920	1840	3410	2690	3120	3070	2.7	8.4
55	12–6	0.78	0.88	5	25	1750	1820	1840	1800	3350	3280	3770	3470	2.1	5.9
56	12–6	0.78	0.82	5	42.5	1870	1860	1710	1810	3280	3450	3270	3330	3.9	2.3
57	12–7	0.76	0.78	5	32.5	1970	2080	2250	2100	2790	3540	3270	3200	4.8	8.5
58	12–7	0.81	0.86	3	20	2210	2100	2280	2200	3690	3730	3180	3530	2.9	6.7
59	12–7	0.87	0.88	6.5	30	2290	2250	2430	2320	3230	3650	3480	3450	3.0	4.3
60	12–7	0.84	0.84	8	45	1750	1780	1800	1780	2740	2910	3320	2990	1.0	3.4
61	12–7	0.83	0.84	4	17.5	1520	1350	1470	1450	3570	3400	3290	3420	4.3	2.9
62	12–8	0.88	0.78	3.5	15	2160	2010	2460	2210	3540	3290	3300	3380	7.6	3.3
63	12–8	0.85	0.78	6	45	1920	1960	1820	1900	3140	3150	3080	3120	2.8	9.6
64	12–8	0.86	0.82	3	15	2220	2040	1910	2060	3380	3090	3330	3270	5.3	3.6
65	12–10	0.87	0.78	3	10	2390	2370	2320	2360	3930	3710	3950	3860	1.1	2.7

* Taken from forms about 1½ hours after depositing.

3 and 4 for the Newark Bay tests. Individual strengths are given in the order of increasing magnitude in Fig. 2.

For the Camden Tests the daily averages for the 1: 2: 4 concrete give a minimum strength of 1660 lb. per sq. in. and a maximum of 2880 lb. These strengths respectively are 24 per cent below and 32 per cent above the average (2190 lb. per sq. in.) of all the strengths. For only 2 days' work was the strength more than 20 per cent below the average and for 3 days' work was it more than 20 per cent above the average; or on only 5 days did the tests give average strengths varying more than 20 per cent from the general average.

TABLE 3.—COMPRESSION TESTS OF 1:2.4:3.6 CONCRETE AT NEWARK BAY (*continued*).

Batch No.	Date Made (1923)	Water Ratio	Void Cement Ratio	Slump, in.	Flow, per cent	Compressive Strength, lb. per sq. in. 7 Days (1)	(2)	(3)	Aver.	28 Days (1)	(2)	(3)	Aver.	Mean Variation of Strength, per cent 7 Days	28 Days
66	12–10	0.76	0.72	5	30	2290	2120	2160	2190	3830	3220	3810	3620	3.1	7.4
67	12–10	0.78	0.78	2.5	10	2430	2300	2260	2330	4480	3700	3780	3990	2.9	8.3
70	12–11	0.85	0.82	2	..	2350	2310	2490	2390	4300	3350	4240	3960	3.0	10.4
71	12–11	0.92	0.84	3	15	2330	2470	1520	2110	3210	3490	2780	3160	18.5	8.0
72	12–11	0.78	0.82	0.7	5	2170	1990	1840	2000	3140	2710	2480	2780	5.6	8.8
73	12–12	0.87	0.82	4.5	20	2690	2440	2230	2450	3390	2670	3170	3080	6.4	8.8
74	12–12	0.88	0.80	2	15	2530	2190	2150	2290	3800	3470	3720	3660	7.0	3.6
75	12–12	0.88	0.82	2.5	15	2950	2510	3130	2860	4970	4160	4570	4570	8.3	5.9
76	12–12	0.88	0.82	4.5	25	2800	2520	2670	2660	3550	3890	3480	3640	3.8	4.6
77	12–12	0.85	0.84	4	20	1820	1670	1720	1740	2710	2920	2480	2700	3.3	5.6
78	12–13	0.81	0.82	5	22.5	2350	2410	2110	2290	3310	3150	3210	3220	5.2	1.8
79	12–13	0.78	0.70	1.5	15	2230	1990	1980	2070	3760	3540	3210	3500	5.3	5.6
83	12–20	0.83	0.78	1.5	10	2520	2770	3090	2790	4050	3600	3680	4110	7.1	7.9
84	12–20	0.78	0.70	3.5	5	2350	2290	2390	2340	3910	3990	4570	4160	1.6	6.7
85	12–20	0.78	0.70	1	10	2970	3190	2850	3000	3510	4230	4440	4060	4.1	9.2
86	12–21	0.83	0.72	1.2	2.5	1680	1810	1580	1690	3530	3350	3040	3310	4.7	5.4
87	12–21	0.77	0.74	2.2	15	1960	1980	1910	1950	3610	3320	3890	3610	1.4	5.5
88	12–22	0.82	0.74	1.5	5	1810	1780	1660	1750	3150	3530	3410	3360	3.4	4.3
89	12–26	0.84	....	1.25	5	1290	1350	1290	1310	....	....	....	....	2.1	...
90	12–27	0.78	0.72	1.0	5	1750	1370	1870	1660	3650	3250	3280	3390	11.9	5.0
91	12–27	0.78	0.76	2.0	10	2050	2010	1840	1970	3390	3360	1750	2830	4.2	26.0
92	12–28	0.68	0.74	2.0	5	2180	2110	2330	2210	3230	3110	3070	3140	3.8	2.0
93	12–28	0.66	0.64	1.0	2.5	2500	2470	2450	2470	3740	3370	3610	3570	0.7	3.8
Average......		0.79	0.79	3.5	22.0										

The average strength of the 1: 1: 2 concrete (80 cylinders) at Camden was 3930 lb. per sq. in. at 28 days. The minimum daily average was 2480 lb. per sq. in., Fig. 1, and the maximum was 4640 lb. per sq. in. Only 1 day's work gave strengths more than 20 per cent below the average, and no day's strength was more than 20 per cent above. An explanation for the lower strengths obtained for the latter part of the work is found in the fact that the quantity of cement for the nominal 1: 1: 2 mix was reduced approximately 10 per cent after about June 1. This change in cement content is indicated in Fig. 1.

Only 10 per cent of all specimens (both 1 : 2 : 4 and 1 : 1 : 2) tested at 28 days gave strengths below 80 per cent of the average. Fig. 2 shows that only 5 per cent of the 1 : 2 : 4 specimens and none of the 1 : 1 : 2 specimens gave strengths below 80 per cent of the compressive strength used as the basis of the structural design.

For the *Newark Bay* tests the nominal 1 : 2.4 : 3.6 mix was used throughout, except in the caisson bottom and in the top of the pier, which formed the bridge seat. The average strength at 28 days of the 228 concrete cylinders of this mix was 3150 lb. per sq. in. That at 7 days was 2010 lb. per sq. in. The minimum daily average for the 28-day tests was

TABLE 4.— COMPRESSION TESTS OF 1:2:3 CONCRETE AT NEWARK BAY.

Compression tests of 6 x 12-in. cylinders made in field.
Aggregate: Sand and pebbles from near Northport, Long Island; furnished by Henry Steers, Inc. Portland cement was used.
Mix, 1:5 damp and loose, separated volumes (6-bag batches).
Cylinders removed from molds and stored in damp sand indoors after about 24 hours.
Samples selected at mixer from 1 cu. yd. batch.
Specimens cured in damp sand until test; tested damp.

Batch No.	Date Made (1923)	Water Ratio	Void Cement Ratio	Slump, in.	Flow, per cent	Compressive Strength, lb. per sq. in.								Mean Variation of Strength, per cent	
						7 Days				28 Days					
						(1)	(2)	(3)	Aver.	(1)	(2)	(3)	Aver.	7 Days	28 Days
17	11–13	0.77	0.67	2.0	..	2810	2380	2390	2530	3810	3800	3180	3600	7.5	7.7
18	11–13	0.77	0.72	3.75	..	2220	2120	2000	2110	3680	3460	3320	3490	3.6	3.7
29	11–19	0.77	0.82	1.5	15	3410	2810	3330	3190	4600	3840	4390	4280	7.9	6.8
30	11–19	0.77	0.80	4.5	30	2160	2040	2210	2140	3540	3180	2830	3180	2.9	7.5
31	11–19	0.71	0.80	5.0	25	2860	2510	2560	2640	4360	3860	3790	4000	5.4	5.9
35	11–20	0.79	0.79	6.0	13	2020	2120	2120	2090	3040	3200	3520	3250	2.1	5.4
36	11–20	0.77	0.77	6.0	12.5	2480	2440	2520	2480	3750	4130	4120	4000	1.1	4.2
68	12–10	0.72	0.70	3.0	..	2450	2360	2310	2370	3750	3850	3820	3810	2.1	1.0
69	12–10	0.70	0.67	2.0	..	2340	2800	2860	2670	4180	4730	4350	4420	8.1	4.7
80	12–13	0.68	0.60	6.0	25	1610	1620	1560	1600	2800	2450	2610	2620	1.4	4.6
81	12–13	0.70	0.67	2.5	25	2910	2530	3870	2770	3650	4030	4870	4180	5.8	10.9
82	12–13	0.66	0.68	5.5	27.5	2810	2730	3000	2850	4120	4460	4000	4190	3.6	4 2
94	12-28	0.57	0.62	2.5	10	2850	3900	3460	3130	4270	4340	4390	4330	6.9	1.0
Average......		0.72	0.72	3.9	20				2500				4100	4.5	5.2

1670 lb. per sq. in. and the maximum was 4110. Minimum and maximum daily average strengths for the 7-day tests were 1310, (the strength of 650 lb. per sq. in. for Nov. 5 in Fig. 1 was from 3-day tests) and 2710 lb. per sq. in. respectively. These minimum strengths are for days on which only one batch was sampled. Here, as in the Camden tests, the variations in strength seem to be largely accidental.

Water-Cement-Ratio and Strength.—For the Camden tests the average water-ratio for the 1 : 2 : 4 concrete was about 1.0 and for the 1 : 1 : 2 concrete about 0.6. The moisture content of the sand ranged from about 3.6 per cent to 7.9 per cent and that of the pebbles from about 0.5 to 4.8 per cent. In Fig. 3 the strength has been plotted against the water-ratio for both the 1 : 2 : 4 and the 1 : 1 : 2 concrete. The points are scattered, but the average curve shows a definite relation. Much of the variation in

strength shown on this diagram may be due to variation of other factors than the water-ratio, and the water-ratio itself may be in error.

In Fig. 3 two curves have been drawn, one based on the equation $f_c = \frac{14000}{.7^x}$ and the other $f_c = \frac{14000}{9^x}$, in which f_c is the compressive strength at 28 days and x is the water-ratio (an exponent). It has been found generally that within the range of workable concrete the first equation represents fairly well the average strengths to be expected. The second curve forms a part of the basis for the calculation of the proportions given in Appendix 16, of the forthcoming Joint Committee Specifications. It was the purpose in preparing those tables to indicate the least strengths which should be expected from the corresponding mixes. This purpose was fairly well realized in these tests. Only about 12 per cent of all of these strengths were below those indicated by the latter equation.

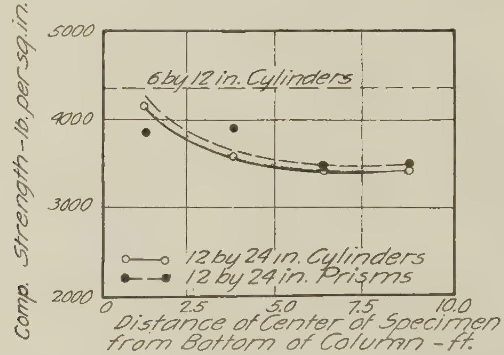


FIG. 5.—COMPRESSIVE STRENGTHS OF SPECIMENS CUT FROM COLUMNS.

Slump and Flow.—The slump and flow varied fairly consistently with each other. The indication is that the slump measured fairly accurately some property of the concrete, but that the strength was more or less independent of that property. Where all other factors than water and cement have been rigorously controlled a distinct relation has been found between strength and slump or flow. Therefore, if there is a marked variation in slump for which there is not a corresponding or a consistent variation in strength there must be a marked variation in some other factors which do not within themselves affect the strength. For example, the quantity of fine material in the aggregate may vary.

The lack of relation between slump and strength seems to be no reason for abandoning the measurement of slump, but rather a reason for controlling more rigidly the measurement of water and aggregate so as to render less likely the occurrence of large and erratic variations of strength and workability.

Effect of Age on the Strength of Concrete.—For the 1: 2: 4 concretes tested at Camden and at the Bureau of Standards the strengths at 7 days were about 1000 lb. per sq. in. and the strengths at 28 days and 3 months were approximately twice and three times as great. The 1: 1: 2 concrete gave strengths at 7 days of about 2000 lb. per sq. in. and smaller proportionate increases in strength at later ages.

A study of the 7 and 28-day strengths suggested a relation which might afford a basis for estimating the 28-day strength of concrete from the results of 7-day tests. The value of such information will be obvious. An examination of published test results showed a relation between the 7-day and the 28-day strengths which is well represented by the equation $f_c = f + 30\sqrt{f}$, in which f_c is the 28-day strength and f is the 7-day strength. The data of the Camden and Newark Bay tests would not be sufficient basis for the derivation of this equation, but it represents their trend quite accurately. The equation was derived from consideration of a large number of tests of concrete having a wide range of mix, consistency, grading of aggregate, and inert powdered admixtures.

Comparison of Strength of Field Cylinders with that of Concrete in Structures.—The strength of the cores drilled from the concrete slabs and of the 6 x 12-in. cylinders made from the same batches are shown in Fig. 4. At 28 days in the Camden tests and 3 months in the Newark Bay tests the cores gave, in general, about the same strengths as the companion cylinders. There was little difference in uniformity of strength in the three sets of specimens. At three months the cores made at Camden gave strengths about 14 per cent lower than those of the cylinders. This probably may be accounted for by the difference in curing conditions. The cores were cured in the air of the Laboratory. Those made at Camden were coated with paraffin. The cylinders were cured in damp sand until they were tested. Little difference can be seen between the two sizes of cores so far as their value for the Camden tests is concerned. In the Newark Bay tests where larger aggregates were used a greater difference might have been found if the smaller cores had been used.

Data of the tests of cylinders and prisms sawed from the concrete columns are given in Fig. 5. The specimens sawed from the columns showed lower strengths than did the 6 x 12-in. cylinders made from the same batch. Fig. 5 shows the strength of the specimens cut from the bottom portion of the columns to be greater than that for those from the upper part.

Concluding Remarks.—The averages of the strengths found at Camden and Newark Bay were as great or greater than the strengths which the mixes used might be expected to yield. They were greater also than the strengths used as a basis of the design. The average strength for the Camden 1: 2: 4 concrete was greater also than the corresponding strength of laboratory concrete made from materials shipped from Camden to the Bureau of Standards.

Generally not over 10 per cent of the specimens of any mix gave strengths less than 80 per cent of the average strength. Laboratory tests do not usually show much greater uniformity.

Therefore, insofar as general results may be predicted from these tests, they indicate that it is possible to meet requirements based upon the Tables of Proportions in the Joint Committee report or based upon average strengths shown by preliminary laboratory tests of specimens using materials from the job under consideration. This assumes that a tolerance of about 10 per cent of the strengths falling below 80 per cent of the average strength is satisfactory.

DISCUSSION OF THE RESULTS OF TESTS OF JOB CONCRETE.

By Walter P. Bloecher.*

Since the convention of a year ago the center of interest in the research field of concrete has been transferred from the laboratory to the construction job. We have witnessed the tests by laboratory experts at Camden, at various operations in New York, at the Newark Bay Bridge, and at the University of Illinois Stadium; not to mention investigators at numerous other construction operations. The principles laboriously evolved have been applied to field practice, with a stronger and more uniform product as the goal.

While the complete answer to the problem of better concrete is yet unknown, it is certain that the activities of the past year have contributed materially to that clarification which must precede the solution of every involved question. As one privileged to follow many of these field tests as a close witness, I venture to present in this brief outline some thoughts bearing on this subject.

TESTING TECHNIQUE.

A. *Molded Cylinders.*—I think I speak the mind of the average construction man when I say that his faith in concrete strength tests has in the past been largely lost by reason of the wide and unaccountable variations in test results which he obtained. Considerable differences in test specimens taken from *different* batches could perhaps be understood; but when the three test cylinders from the *same* batch showed spreads as high as 40 or 50 per cent of the average, which became a common experience, it was but natural to consider the yardstick faulty.

We can now say that a large part of this variation was due to an important variable which was not properly understood, and which apparently can be largely eliminated. I refer to the technique governing the testing operations. In Fig. 1 the mean deviation of the three cylinders from the batch average is computed for each batch tested; these values are then arranged in the order of magnitude and plotted as percentages into a smooth curve. The Camden cylinders and cores are compared in this manner with similar results of job testing experienced on a typical 1922-23 operation designated "B" at which no expert testing talent was available. It happens this second operation was also in the Philadelphia district and the same materials were used as at Camden. The contrast is striking, the deviation from the batch average for the 1 : 2 : 4 Camden cylinders is less than one-third that at Operation B; in other words, the testing technique at Camden was over three times as accurate as at Operation B. Again as against a mean deviation for the entire series of 202 lb.

---

*With Stone & Webster, Inc., Boston, Massachusetts.

or 11 per cent, for this operation, we find the Camden tests gave only 69 lb. or 3.19 per cent, while the mean deviation of the drilled cores averaged 152 lb., or 6.6 per cent. The results obtained by testing experts at the Newark Bay drawbridge, called Operation A, is also shown, where the average deviation was 198 lb. or 5.9 per cent.

This improvement is mainly due to refined methods of testing; and the value of a properly made cylinder as a true measure of the material it represents is more certainly established. At Camden it was found, for instance, that the batch into which the sweepings from the cement plat-

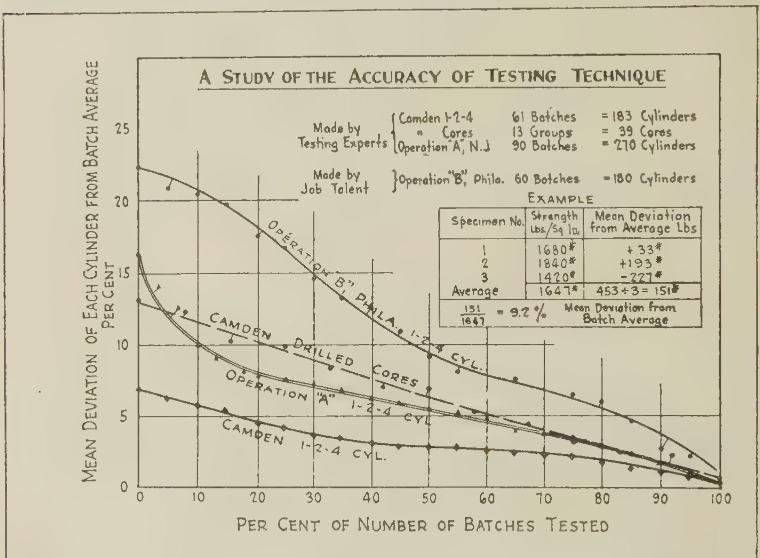


FIG. 1.

form were added, responded instantly by showing higher strength; and that the first 1 : 2 : 4 batch following a 1 : 1 : 2 batch was invariably stronger by reason of the rich mortar retained in the mixer. Again, the day the gravel supply ran low and the storage area was scraped for aggregate gave the lowest tests of the series.

Concrete is a sensitive material; and making tests of it is a science. The need for great refinement is apparent when we contemplate that a three-minute delay may change the slump test several inches; that a cylinder improperly capped may show a loss in strength upward of 50 per cent; that a change in the method of sampling may largely change the strength shown by the cylinder; or that one molded slightly out of plumb may show an improperly low strength, not to mention a host of other influences.

Our immediate problem is to codify the lessons learned in the art of testing so they may be available to the ordinary job organization. We can then expect to see intelligent strengths from every-day tests and confidence in the results on the part of the job organization, which should precede any important change in manufacturing methods.

B. *Drilled Cores vs. Molded Cylinders.*—Nothing new seems to have been learned from the core specimens drilled at Camden, except that their strengths agreed quite closely with the corresponding molded specimens. In retrospect, it appears to the speaker that nothing new was to have been expected, for the reason that a properly made core specimen must give substantially the same strength as a properly molded cylinder from the same batch, and can differ mainly in having the added variable of uncertain curing conditions. The curve on Diagram No. 1 indicates this added variable for cores in the wider departure from the average of individual cores from the same batch. Core specimens appear to be practicable in building work only in special situations, or to check the actual structure against apparently erroneous variations in cylinder tests. Mention might be made here of an actual experience where cylinder tests indicating strengths under 1000 lb. gave concern to the management, and cores, later cut from the same run of concrete showed strengths around 3000 lb.

MANUFACTURING TECHNIQUE AND UNIFORMITY OF TEST RESULTS.

The general problem under discussion has two aspects, first, the technique of testing and, second, the technique of manufacture. The laboratory is responsible for the former, and the construction industry as a whole is responsible for the latter, except as it may permit itself to be governed by laboratory principles. In most of the experiments lately made, the testing technique was the principal new force introduced, and the technique of manufacture was not substantially changed from ordinary commercial practice. We have discussed the accuracy of testing, and now pass on to the uniformity obtained from our manufacturing methods.

When we come to examine the uniformity obtained at Camden from batch to batch, the results have shown nothing particularly new. Assuming, as we now may, that properly made cylinders are fair indices to the strength of the batch they represent, we still find extreme variations running from 1160 lb. to 3200 lb. between successive batches. On Diagram No. 3, the 28-day Camden tests have been superimposed on the results of a series of job tests presented to the 1923 convention. This method of analyzing the results consists in arranging them in the order of their strength and plotting the succeeding values into a smooth curve which can be called a progressive strength curve.

We find that while varying about as widely as most previous test results, the Camden strengths do lie in a higher zone. It happens that Curve D* gives results of concrete which was made two years previously

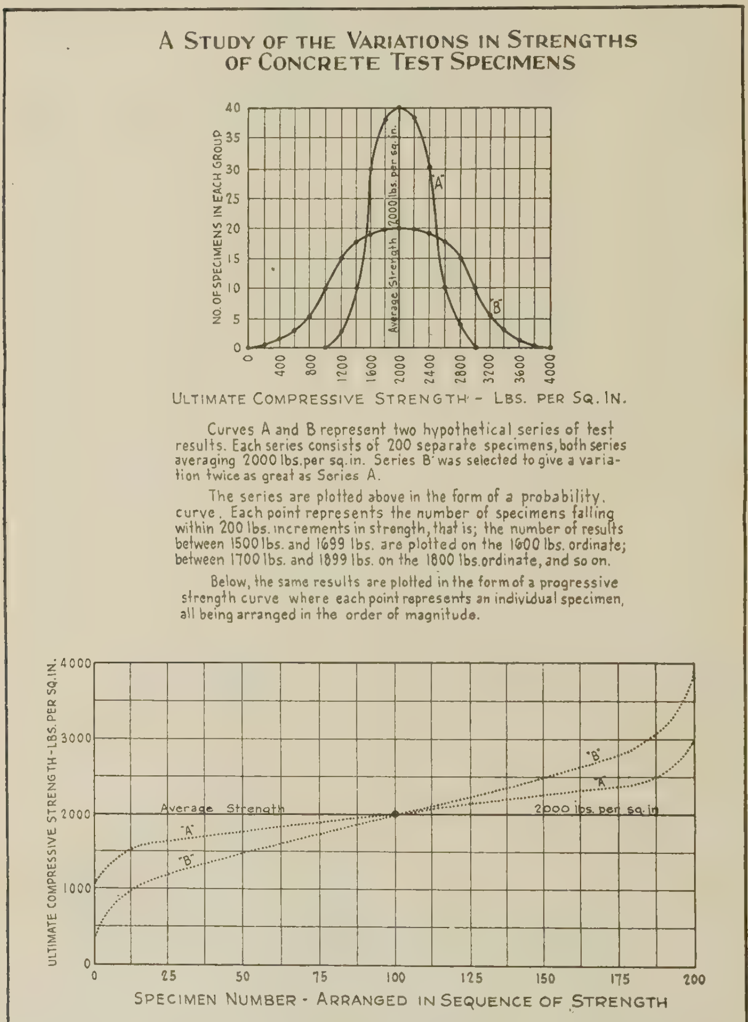


FIG. 2

by the same superintendent, the same concrete foreman and the same mixer as were on the job at Camden; and offers a basis for an indirect comparison. We believe the improvement which the Camden tests show is due to

*See "An Analysis of the Variables in Concrete from the Construction Standpoint," 1923 Proc. Am. Conc. Inst.

Fig. 3

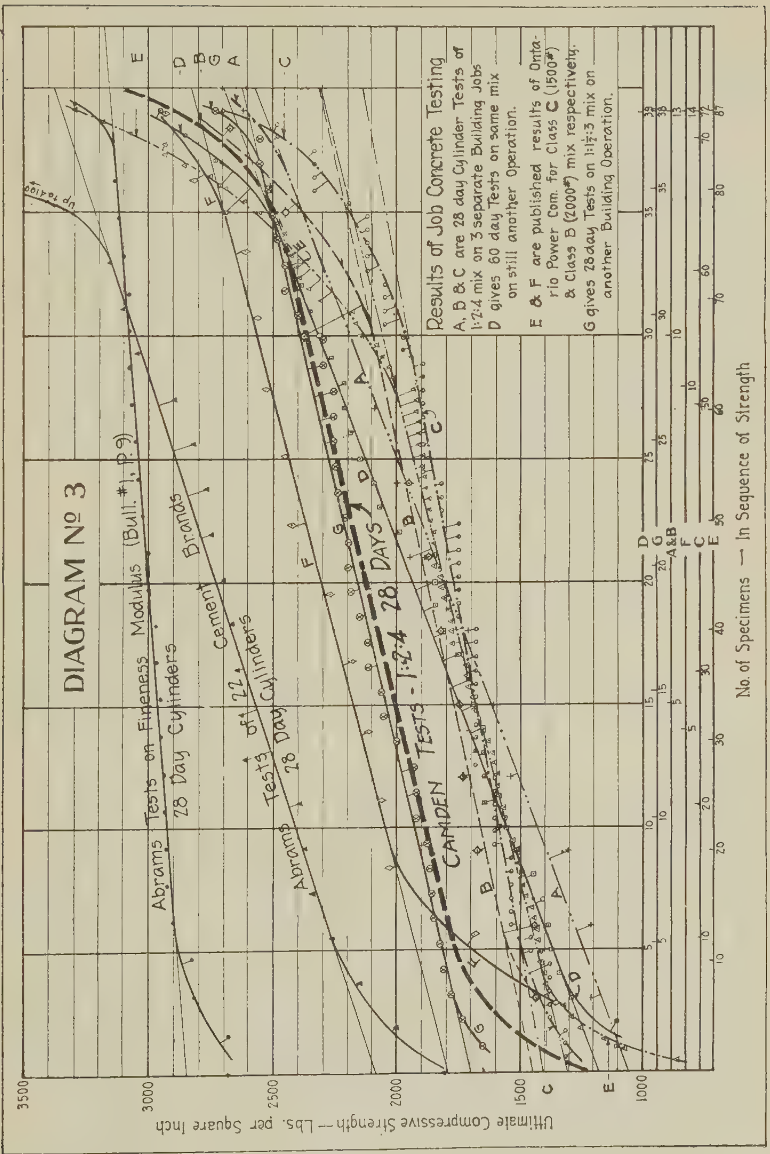


DIAGRAM No 3
Abrams Tests on Fineness Modulus (Bull. #1, P. 9)
28 Day Cylinders
Abrams 28 Day Tests of 22 Cement Brands
Cylinders
CAMDEN TESTS - 1:2:4 28 DAYS
up to 4100
Results of Job Concrete Testing
A, B & C are 28 day Cylinder Tests of 1:2:4 mix on 3 separate Building Jobs
D gives 60 day Tests on same mix on still another Operation.
E & F are published results of Ontario Power Com. for Class C (1500#) & Class B (2000#) mix respectively.
G gives 28 day Tests on 1:1½:3 mix on another Building Operation.
Ultimate Compressive Strength — Lbs. per Square Inch
No. of Specimens — In Sequence of Strength

two things; first, the superior aggregates available in the Philadelphia market, and, second, the superior testing talent at Camden.

It is probable that all such variations as were found from batch to batch at Camden represent real differences in the quality of the material and that our finished product is really quite spotty. The problem lies in finding the causes for these differences and applying practical remedies. It is my belief that many of these remedies lie beyond the contractor's control, and that we should not lose sight of the responsibilities of the cement manufacturer, the aggregates dealer, the engineer, and the owner. It is difficult to see where any substantial change can be made by the contractor in his construction methods, except along three principal lines: (a) in the measurement of water (taking into account entrained water in aggregates), (b) in the measurement of aggregates, and (c) in curing. Even when these have been reduced to a certainty we must still expect considerable variation in the results. For instance, at the Newark Bay Bridge tests, where the mix was designed by experts and accurate water control and volumetric measuring devices were used, this same lack of uniformity was still present. Also at the Ward Baking Co. operation in New York, where the inundation method of sand and water measurement were used, no particular improvement in uniformity was obtained.

In the upper part of Fig. 2 are given probability curves which outline two hypothetical series of cylinder test results. "A" gives a rather uniform series approaching closely to the grand average and "B" gives one having departures from the average twice as great as "A." When these results are expanded to the form of the Progressive Strength curve, they take the shapes shown in the lower part of the figure. It is interesting and important to note the close similarity between the shape of these hypothetical curves, and actual experiences obtained in job testing, as shown in Fig. 3. It strongly indicates that the variations experienced follow some law of probability.

It has been found too that many factors affecting concrete strengths individually give variations throughout the life of a job which when plotted in the sequence of magnitude, also take the shape of the Progressive Strength curve. Cement strengths,* fineness moduli of sand and gravel,† water content of sand,† impurities in aggregates, flow table† and slump tests† have been particularly noted. The variations in strength experienced can thus be explained as due to a large number of such factors which varying slightly from batch to batch produce an infinite variety of combinations which give the wide range of results obtained. Good consistent results will come only with refinement in all operations, and must include the manufacturer of the cement, the aggregates dealer, the contractor, the engineer, and the U. S. Weather Bureau.

---

*See paper, "An Analysis of the Variables in Concrete," Proc. A. C. I., 1923.

† See paper, "Field Tests of Concrete," by Ahlers and Walker, Proc. A. C. I., 1924.

Parenthetically, I venture to state, as my personal opinion, that it is about time we stopped thinking and began practicing the accurate measurement of our aggregates, either by volume corrected for moisture content, or by weight. Also, knowing how important curing is, it is not clear why the building contractor gives less attention to moist curing his building frame than the laboratory man gives to his cylinders, or the highway builder gives to his concrete pavement.

The range in values when stated as "80 per cent of the specimens came within 20 per cent of the average" is a rather favorable statement of the facts. It means, for example, that four-fifths of the results ran from

TABLE 1.—COMPARISON OF UNIFORMITY OF TEST RESULTS ON VARIOUS CONSTRUCTION OPERATIONS.

Job.	Age.	Mix.	Extreme Variations.				Straight Line Variations (Extremes Omitted).				Mean Variation from Grand Average, per cent.
			High.	Low.	Average.	Per cent Spread of Average.	High.	Low.	Average.	Per cent Spread of Average.	
TESTS BY EXPERTS.											
Camden Cylinders	28	1:2:4	3200	1160	2150	90.5	2600	1650	2150	44.2	14.7
Camden Cylinders	28	1:1:2	5100	3000	4100	51.2	5050	3200	4100	45.1	13.0
Camden Cores (4½ in. dia.)	1–2 mo.	1:2:4	2900	1660	2350	52.8	2850	1800	2350	44.7	10.7
Operation A–New Jersey	28	1:2:4	4980	1580	3200	106	4000	2400	3200	50.0	10.8
TESTS BY JOB TALENT.											
Operation B–Philadelphia	28	1:2:4	4270	770	1835	190	2600	925	1810	92	....
Operation D–Baltimore	28	1:2:4	2950	1102	1940	95	2650	1200	1925	75	21.4
Operation C–New York City	28	1:2:4	2700	1312	1797	77	2120	1400	1760	41	9.6
Young's Class B Ontario Power Development	28	....	2937	1365	2315	68	2800	1870	2335	40	14.4

1600 lb. to 2400 lb., the other fifth being anywhere from 1160 lb. to 3200 lb. The average deviation from the mean value for the entire series tends also to create too rosy a picture of the results. The speaker believes a better modulus of uniformity is to be found in expressing the slope of the progressive strength curve as a function of the average strength. It is proper to omit the extremely low as well as the high results and confine ourselves to the straight line tendency.

Table 1 gives such a comparison in statistical form between the uniformity obtained at Camden and that obtained on jobs previously reported to the Institute.* It shows for example that the so-called straight-line variation obtained at Camden which was about 45 per cent, was equalled on several other previous operations.

*See paper, "An Analysis of the Variables in Concrete," Proc. A. C. I., 1923.

A PROPOSED TOLERANCE RULE.

Tolerance rules so far proposed have been open to the serious objection that one could never foretell whether one's results were falling within the rule until all job testing was completed. A method by which we can know currently whether the results are falling within proper limits is suggested as follows: Knowing the yardage in a proposed operation, the number of cylinders to be taken can be defined as so many per 100 cu. yd., let us say three. Then knowing the strength desired, we can construct a hypothetical progressive strength curve showing the variations or tolerances to be permitted. On this line a space is provided for every cylinder test to be made, and as each specimen is broken, the result can be plotted on this curve nearest the point of equal value. Just so long as vacant spaces still exist lower in value than any particular one obtained, we can be sure we are still on the safe side. We should, of course, expect a liberal distribution of points currently both above and below the average line. This method takes into account the probabilities of extremely high or low results and discounts them accordingly.

If we should find in the midst of a job, for example, that we have obtained some low values, when all low spaces have been filled by previous tests, we know at once that our ultimate results will fall short of the prescribed goal, and means can be taken to correct conditions say by increasing the cement factor. Again, if we should find our results taking up the higher spaces, we can feel confident that our cement factor could be safely reduced. The tolerance rule thus consists in defining the characteristics of the hypothetical line, which present experience indicates to be one having a straight line spread of about 45 per cent. As our manufacturing technique improves, the uniformity of results will improve with it, and this will be reflected in a flatter curve.

It may be of interest to remark that the job organization followed the tests with much interest and an open mind. They were glad to find out definitely the strengths of the concrete they were making and the quality of the raw materials they were receiving, and they felt some pride in the high values obtained. But they do not feel that the slump or flow table tests can replace the experienced man who judges the water content from the appearance of his mixture.

The concrete industry is more than ever in a transitional period, and as these various difficulties are overcome, one by one, we can expect to see the unit fibre stress for ordinary structural concrete raised to say 800 lb., with its resultant saving in construction costs. The problem in the recent past has been considered mainly as one finding the optimum mix and holding down the water content; while a host of other factors, of lesser individual importance to be sure, have been side-tracked. We should not be misled by the thought that there is a single cure-all, or that the goal can be reached by anything short of refinement in all the ramifications of the industry, and we should remember that the cost of these refinements must not be handled as a tax on the industry, but must be justified by

ultimate over-all economy. In the meantime, we are assured from these test results that our present product is in general good, in fact better than we had reason to anticipate.

It seems entirely fitting that acknowledgment be made at this time of the high order of ability with which Mr. Slater and Mr. Walker attacked the problem at Camden. They deserve much credit for what has been demonstrated, as do all other co-operating agencies, and it is a pleasant commentary to add that the fine co-operative spirit which prevailed augurs well for continuing in the field the efforts to improve our quality of concrete.

TESTS OF IMPURE WATERS FOR MIXING CONCRETE.

By Duff A. Abrams.

SUMMARY AND CONCLUSIONS.

Strength tests of portland cement concrete were made at ages of 3 days to 2 1/3 years using mixing waters of a wide range of types, many of which were thought to be unsuitable for use in concrete. Sixty-eight samples of water were tested in two different investigations; 52 samples were collected from different sections of the country; 14 were from the Chicago district. Among the waters tested were sea and alkali waters, bog waters, mine and mineral waters, waters containing sewage and industrial wastes, and solutions of common salt. Tests of fresh waters (including distilled) were made for purpose of comparison. About 6000 tests are included in this report. Reference is made to a number of other investigations on related subjects.

Series 137.—50 samples of water were used in 1: 4 concrete of relative consistency 1.10 and tested after curing under the following conditions:

(a) Moist room, tested at ages of 3 days to 2 1/3 yr.,
(b) Moist room 28 days, then in air of laboratory, tested at 3 mo. to 2 1/3 yr.

Series 138.—18 samples including sewage and trade wastes from the Chicago district, were used in concrete cured in moist room as follows:

(a) 1: 7, 1: 5, 1: 4, 1: 3 and 1: 2 mixtures, relative consistency 1.00, tested at 28 days.
(b) 1: 4 concrete mix, relative consistency 0.90, 1.00, 1.10, 1.25, and 1.50, tested at 28 days.
(c) 1: 4 concrete mix; relative consistency 1.00, tested at 3, 7, and 28 days, 3 mo., 1 and 2 1/3 yr.
(d) 1: 3 standard sand mortar, tension tests of briquets and compression tests of 2 by 4-in. cylinders at ages given in (c).

Concrete and mortar tests were made in accordance with standard methods. In general, 5 to 10 concrete specimens, and 5 mortar specimens were made in a set on different days.

Time of setting and soundness tests of cement were made with each sample of mixing water.

The principal conclusions from these tests are:

(1) In spite of the wide variation in the origin and type of the waters used, and contrary to accepted opinion, most of the samples gave good results in concrete. This result seems to be due to the fact that the quantity of injurious impurities present is quite small.

The following samples gave concrete strengths below the strength-ratio of 85 per cent which was considered the lower limit for acceptable mixing waters: Acid waters, lime soak from tannery, refuse from paint factory, mineral water from Colorado, and waters containing over 5 per cent of common salt.

(2) The quality of a mixing water is best measured by the ratio of its 28-day concrete or mortar strength to that of similar mixes with fresh water. While the lowest permissible strength-ratio is a matter of judgment, waters giving strength-ratios which in general fall below 85 per cent should be considered unsatisfactory; if only isolated tests are made, 80 per cent should be the limiting value. The time-of-setting test appears to be an unsafe guide as to the suitability of a water for mixing concrete.

(3) Neither odor nor color are any indication of quality of water for mixing concrete. Waters which were most unpromising in appearance, gave good results. It may safely be said, however, that any natural water which is suitable for drinking can be used without question for mixing concrete.

(4) Distilled water gave concrete strengths essentially the same as other fresh waters.

(5) Bog waters which were thought to be unsuitable for mixing concrete generally contained only small quantities of foreign materials and gave good results. The strength-ratios for the individual samples were seldom below 90 per cent.

(6) Sulphate waters produced little or no ill effects until a SO_4 concentration of about 1 per cent was reached. For a concentration of 0.5 per cent the average reduction in strength was about 4 per cent; a concentration of 1 per cent was required to produce a reduction in strength of more than 10 per cent.

(7) Concrete mixed with sea water (about 3.5 per cent salts, mostly sodium chloride) and cured in the moist room gave higher strengths than fresh-water concrete at ages of 3 and 7 days; at 28 days and over, the strength-ratios for sea water ranged from 80 to 88 per cent. Air-cured concrete mixed with sea water was lower in strength than similar fresh-water concrete at 3 mo., but showed a recovery in strength at later ages and gave strengths equal to that obtained with fresh water. (In spite of the satisfactory strength results, it seems unwise to use sea water in reinforced-concrete construction, particularly in the tropics on account of danger of corrosion of reinforcement.)

(8) Synthetic sea water gave concrete and mortar strengths similar to natural sea water.

(9) Concrete mixed with water from the Great Salt Lake (about 20 per cent sodium chloride) gave strength-ratios from 65 to 77 per cent at ages of 28 days and over. This water is not satisfactory for mixing concrete, unless allowance is made for about 30 per cent reduction in the assumed strength.

(10) Water from Devil's Lake, North Dakota (0.15 per cent sodium sulphate and 0.15 per cent sodium chloride), gave normal concrete strengths and showed no ill effects.

(11) Water from Medicine Lake, South Dakota (3.5 per cent solution of sulphates, largely magnesium; SO_4 concentration 2.8 per cent), gave strengths similar to that obtained with sea water. The lowest strength-ratio was 84 per cent.

(12) Waters from drains and small streams in sulphate districts gave satisfactory strengths at ages up to 2 1/3 yr. The lowest strength-ratios were about 90 per cent.

(13) Concrete made with solutions of common salt and cured until test in a moist room showed a slight increase in strength at 3 days for solutions of 10 per cent and less. Solutions of low concentration (1 and 2 per cent) also showed a slight increase in strength at 7 days; after 7 days, however, all concentrations gave material reductions in strength. Strength-ratios as low as 60 per cent were found for a 20 per cent solution at early ages and for 10 per cent and 15 per cent solutions at the later ages.

Concrete made with salt solutions and cured in the moist room for 28 days, then in air, gave results at 3 mo. almost identical with that obtained for moist-room curing. For this curing condition, the addition of salt reduced the strength at 1 yr. about 12 per cent; at 2 1/3 yr. there was no reduction in strength. The apparently conflicting results for moist-room and air-curing have not been explained.

(14) The use of common salt for the purpose of lowering the freezing point of the mixing water during cold weather should not be permitted; 5 per cent of salt lowers the freezing point of water about 6° F., but reduces the strength of concrete about 30 per cent.

(15) Mine and mineral waters gave good results in concrete, with the exception of a carbonated mineral water from Colorado which gave strength-ratio as low as 80 per cent. Pumpage waters from coal and gypsum mines also gave good results in concrete.

(16) Water containing sanitary sewage gave essentially the same concrete strength as fresh water. Water from the Illinois River, which carries sewage from Chicago, gave strength-ratios at 28 days and 3 mo. of 83 and 85 per cent for moist-room curing; for air-curing strength-ratios ranged from 92 to 102 per cent.

(17) Waters containing refuse from oil refineries gave erratic strengths. These samples generally gave strengths near normal, but in some cases material reductions in strength were found. Setting time of cement with one water sample was, initial 10 hr., final 42 hr.

(18) "Bubbly Creek" water, which is highly polluted with wastes from the Chicago Stockyards and gave off an offensive odor showed strength-ratios of about 100 per cent for all ages, mixes and consistencies.

(19) Tannery wastes generally gave reductions in concrete strength; the lowest strength-ratios were about 80 per cent (lime soak water).

(20) Brewery and soap works wastes gave concrete strengths essentially the same as that of fresh water.

(21) Waste from a gas plant and a corn products factory gave good results; the strength-ratios ranged from 90 to 100 per cent.

(22) Paint factory waste water gave strength-ratios ranging from 80 to 90 per cent.

(23) A spent plating bath containing sulphuric acid, after dilution to 10 and 20 per cent of its original concentration, gave strength-ratios as low as 85 per cent for the 10 per cent solution and 74 per cent for the 20 per cent solution. For different consistencies both solutions gave about the same strength-ratios which ranged from 88 to 106 per cent. Lower strength-ratios were obtained with the rich concretes than with the lean.

(24) The strength of concrete mixed with all samples of impure waters showed normal increase at 28 days with additional quantities of cement. The impure waters gave about the same strength-ratios regardless of the mix used in the concrete tests. For the usual range in mixtures (1: 5 to 1: 4) the strength increased about 1 per cent for each 1 per cent additional cement.

(25) There was a marked reduction in strength of concrete with increase in quantity of mixing water for both fresh and impure waters. Increasing the quantity of mixing water 1 per cent reduced the strength of concrete about the same amount as if the quantity of cement were *reduced* 1 per cent. A comparatively slight increase in *quantity* of mixing water produced a greater reduction in concrete strength than that caused by the use of the *most polluted* mixing water that is ordinarily encountered. These tests show the importance of the water-ratio strength relation in concrete which has been pointed out in numerous other reports from this Laboratory.

(26) The effect of impure waters was in general independent of the consistency of the concrete. Acid waters from a spent plating bath gave somewhat higher strength-ratios in the wetter concretes.

(27) The strength of concrete cured in a damp condition at normal temperatures increased with age for both fresh and impure waters. The strength was approximately proportional to the logarithm of the age at test.

(28) The effect of impure mixing waters on the tensile and compressive strength of 1: 3 standard sand mortar at ages of 3 days to 2 1/3 yr. was generally similar to that on the compressive strength of concrete. In the mortar tests a few waters gave somewhat higher strength-ratios and one water gave a somewhat lower strength-ratio than was obtained in concrete.

(29) The percentage of water required for normal consistency of cement when mixed with the impure water was, with few exceptions,

about the same as for fresh waters. Water from Great Salt Lake, solutions of 5 to 20 per cent of common salt, refuse from an oil refinery, Medicine Lake water, and acid water from a spent plating bath, required somewhat higher percentages for normal consistency than fresh water.

(30) The time of setting of portland cement mixed with the impure waters was about the same as for fresh waters; however, there were some notable exceptions. In most instances the samples giving low concrete strength-ratios were slow setting. On the whole the tests show that time of setting is not a satisfactory test for suitability of a water for mixing concrete.

(31) None of the impure waters caused unsoundness of the portland cement when subjected to the standard test over boiling water.

(32) Most specifications for water for mixing concrete are so worded that they would, if strictly enforced, exclude nearly all but rain water and distilled water; these tests have shown that almost any impurity may be present without *necessarily* producing ill effects. The important point is not whether impurities are present, but do the impurities occur in *injurious* quantities?

(33) The effect of sugar and similar compounds was not studied; earlier tests have shown that these compounds are most detrimental and must be avoided.

INTRODUCTION.

Water is one of the essential constituents of concrete, hence it is important to establish the effect of both *quantity* and *quality* of mixing water on the strength and other properties of concrete. Tests made in this and other laboratories have shown the effect of *quantity* of mixing water; the present investigation covers an experimental study of the effect of *quality* of mixing water.*

In many sections of the country the possibility of building concrete structures, especially concrete roads, may depend on the suitability of the local waters which may be contaminated with various impurities; alkali waters, bog waters, and waters containing sewage, mine pumpage, or other trade wastes are characteristic of many important industrial districts.

---

*See following reports for the effect of *quantity* of mixing water:—

Effect of Consistency on Strength of Concrete; Jl. Am. Concrete Inst., Oct.-Nov., 1914.

Strength and Other Properties of Concrete, by Wig and others; Technologic Paper 58, U. S. Bureau of Standards, 1916.

Effect of Grading of Sands and Consistency of Mix upon the Strength of Plain and Reinforced-Concrete, by L. N. Edwards; Proc. Am. Soc. Testing Materials, Part II, 1917.

Strength of Concrete; Its Relation to the Cement Aggregates and Water, by Talbot and Richart; Bulletin 137, Eng. Experiment Station, Univ. of Ill., 1923.

Bulletins 1, 2, 5, 8, and 11 of the Structural Materials Research Laboratory, Lewis Institute, Chicago.

Economic Value of Admixtures, by Pearson and Hitchcock; Proc. Am. Concrete Inst., 1924.

Table 1.—Notes on Water Samples—Series 137.

In general 5 to 10 gallons of each sample were secured for test.
See Table 2 for chemical analyses of water samples.

Sample	Origin of Water Samples
	Group 1.—Fresh Waters.
2, 50	Lake Michigan; Chicago city water supply.
1, 49	Distilled water.
14	Coal City, Ill.; city water supply.
15	Tracy, Minn.; city water supply, from 700-ft. well.
17	Marshall, Minn.; city water supply from wells; high alkali content for potable water.
18	Dundee, Mich.; city water supply; has hydrogen sulfide odor; high alkali content for potable water.
36	Lake County, Ind.; rain water from small pools near Gary.
37	Youghiogheny River, foot of Fifth Street, McKeesport, Pa.
	Group 2.—Minnesota Bog Waters.
19	Pale Face River } Fed by peat bogs or tamarack swamps; yellow color. Probably average conditions for rivers in Northern Minnesota. Representative waters available for concrete work in St. Louis County.
20	Helwig Creek
21	Coloquet River
24	Dunka River
22	Stagnant pool in peat bag. Very little flow; peat floating on surface; yellow color.
23, 25	Sulphur Dam; typical of swamp waters in Northern Minnesota; yellow color.
	Group 3.—Sea Water and Alkali Waters.
3	Sea water from Atlantic Ocean, Wildwood, N. J.
16	Sea water from Atlantic Ocean, Bay Head, N. J. No sewage present.
4	Synthetic sea water; 3.5 per cent solution of salts approximating composition of sea water.
5	Great Salt Lake, Utah; contains about 20 per cent soluble salts and some vegetation.
26	Devil's Lake, N. D.; high in alkali.
27	Marshall, Minn. } From low water flow of tile drains in alkali soils.
28	Roswell. N. M.
29	Montrose, Colo.
31	Muddy Creek, Utah, 3 miles north of Emery; Federal Road Project; high alkali content.
46	Price, Utah; sample furnished by State Road Commission; alkaline.
	Group 4.—Solution of Common Salt.
6 to 11	Solutions of 1, 2, 5, 10, 15, and 20 per cent table salt in Lake Michigan water.
	Group 5.—Mine and Mineral Waters.
12	Montrose, Colo.; mineral water from mineral spring at City Hall Square; highly carbonated.
30	Bisbee, Ariz.; Mine water.
32	Loop Creek, W. Va., Fayette Co.; 24 mines pump drainage into stream above this point.
33	Cambria, Ill.; coal mine water from mine pump discharge.
34	Colpville, Ill.; from mine pump discharge.
35	Fort Dodge, Iowa, Gypsum mine water, from mine pump discharge.
43	Scranton, Pa.; Coal washery waste, from mine pump discharge.
	Group 6.—Waters Containing Sewage and Industrial Waste.
13	Illinois River, near Marseilles; carries sewage from Chicago Drainage Canal.
38	Lakehurst, N. J., Air Station; from creek; hydrogen sulfide odor; yellow color.
39	Monongahela River, below B. & O. Bridge at Wheeling Junction, Pa.; yellow color.
40	Franklin, Pa.; Refuse from oil refinery; black and oily.
42	Endicott, N. Y.; probably tannery waste.
44	"Bubbly Creek," Chicago; waste from Stock Yards; very dark color; scum on surface.
45	Cygnet, Ohio, from Rockford Creek; contains salt and oil scum from oil wells.
48	Allegheny River (low water) } From Franklin, Pa.; probably contains refuse from oil refineries.
41	Allegheny River (high water)
47	Drainage Canal, Chicago; contains large quantity of sewage.

Table 2.—Chemical Analyses of Water Samples—Series 137.

See Table I for further details of origin and nature of samples.
Chemical analyses expressed as parts per million.

Sample	Source	Suspended Matter	Total Solids	Ignited Solids	Silica (SiO_2)	Iron and Aluminum Oxides (R_2O_3)	Calcium (Ca)	Magnesium (Mg)	Sodium and Potassium (Na & K)	Sulphate (SO_4)	Carbonate (CO_3)*a*	Chloride (Cl)
	Group 1.—Fresh Waters.											
2,50	Lake Michigan......	..	150	150	..	..	35	25	..	15	70	5
1,49	Distilled water.......	..	...	...	..	..	..	..	..	..	..	..
14	City water, Coal City, Ill...............	20	1,250	1,120	5	5	95	55	200	370	140	240
15	City water, Tracy, Minn.............	15	1,260	1,150	trace	trace	240	65	40	650	160	5
17	City water, Marshall, Minn.............	low	2,770	2,600	..	..	340	100	400	1,480	210	65
18	City water, Dundee, Mich.............	low	1,920	1,740	..	15	330	125	70	1,060	100	35
36	Lake County, Ind....	35	660	510	10	..	150	20	trace	170	140	25
37	Youghiogheny River.	low	25	15	5	5	1	1	0	0	trace	trace
	Group 2.—Minnesota Bog Waters.											
19	Pale Face River.....	low	115	80	15	5	15	5	5	10	30	5
20	Helwig Creek........	70	115	80	5	5	15	5	5	10	..	..
31	Cloquet River.......	low	145	105	10	15	20	5	5	5	40	5
24	Dunka River........	10	120	60	10	5	10	5	trace	10	20	5
22	Stagnant pool.......	low	195	130	5	10	25	10	trace	trace	70	5
23	Sulphur Dam	250	90	70	..	..	25	trace	..	15	20	10
25	Sulphur Dam........	10	105	40	5	5	10	trace	5	..	10	5
	Group 3.—Alkali Waters.											
3	Sea water...........	100	33,400	33,420	..	..	350	1,000	11,100	2,370	30	18,500
16	Sea water...........	low	33,300	33,300	..	35	550	1,080	10,000	2,460	70	18,600
4	Synthetic sea water..	none	35,000	35,000	..	..	420	1,310	11,100	2,700	70	19,400*b*
5	Great Salt Lake.....	low	227,800	227,800	..	..	1,280	6,330	78,600	14,400	410	126,700
26	Devil's Lake, N. D...	15	4,000	4,000	15	5	20	trace	1,580	1,070	370	910
27	Tile Drain, Minn....	10	1,910	1,820	20	5	260	170	115	1,060	190	5
28	Tile Drain, N. M....	820	3,130	3,130	10	..	560	120	665	1,490	130	140
29	Tile Drain, Colo.....	..	2,490	2,350	10	..	470	110	160	1,520	80	5
31	Muddy Creek, Utah..	240	12,100	12,100	..	2	5	270	3,320	7,700	210	220
46	Price, Utah.........	low	9,430	7,890	..	40	440	510	1,200	5,500	130	65
	Group 4.—Solutions of Common Salt.											
6	1 per cent common salt...............	..	10,200	10,200	..	..	35	25	3,960	15	70	6,110
7	2 per cent common salt...............	..	20,400	20,400	..	..	35	25	7,980	15	70	12,300
8	5 per cent common salt...............	..	52,000	52,000	..	..	35	25	20,400	15	70	31,400
9	10 per cent common salt...............	..	107,500	107,500	..	..	35	25	42,200	15	70	65,100
10	15 per cent common salt...............	..	166,800	166,800	..	..	35	25	65,500	15	70	101,100
11	20 per cent common salt...............	..	223,100	223,100	..	..	35	25	87,800	15	70	135,200
	Group 5.—Mine and Mineral Waters.											
12	Mineral water, Colo..	..	2,140	2,060	5	trace	90	40	670	270	880	100
30	Mine water, Ariz....	25	400	360	5	trace	90	10	20	180	40	10
32	Loop Creek, W. Va..	low	720	680	15	5	70	40	80	450	10	10
33	Coal mine, Ill.......	5	4,640	4,400	15	10	260	180	1,000	2,780	100	60
34	Coal mine, Ill.......	15	2,240	2,120	trace	5	110	50	580	410	250	670
35	Gypsum mine, Iowa..	30	2,660	2,220	25	trace	560	80	80	1,370	90	10
43	Coal washery, Pa....	low	2,700	2,460	..	170	260	170	50	1,780	*d*	60

TABLE 2—CHEMICAL ANALYSES OF WATER SAMPLES.—SERIES 137 (*continued*).

Sample	Source	Suspended Matter	Total Solids	Ignited Solids	Silica (SiO_2)	Iron and Aluminum Oxides (R_2O_3)	Calcium (Ca)	Magnesium (Mg)	Sodium and Potassium (Na & K)	Sulphate (SO_4)	Cqrbonate (CO_3)a	Chloride (Cl)
	GROUP 6.—WATERS CONTAINING SEWAGE AND INDUSTRIAL WASTES.											
13	Illinois River.........	low	970	920	5	trace	100	50	160	220	170	210
38	Lakehurst Air Station	10	450	390	15	50	40	10	10	250	5	5
39	Monongahela River..	40	300	270	5	15	30	10	50	150	..	5
40e	Refusefromoilrefinery	..	...	...	..	..	...	...	...	...	..	...
42	Endicott, N. Y......	250	9,040	7,110	..	..	1,220	trace	1,650	1,000	470	2,200
44	"Bubbly Creek", Chicago...........	low	1,860	1,090	35	35	90	20	390	60	200	650
45	Cygnet, Ohio........	low	4,300	4,020	..	20	290	130	1,190	250	50	2,180
48	Allegheny River (low water)............	..	120	100	..	..	30	5	..	20	30	15
41	Allegheny River (high water)............	..	250	220	5	5	30	60	10	30	70	30
47	Drainage Canal, Chicago..........	low	260	220	15	20	40	10	10	50	80	trace

a Calculated from bicarbonates which were determined by titration.
b 65 parts per million of Bromine also present.
c Analyses calculated from analysis of Lake Michigan water and specific gravity of salt solutions.
d Water is acid; acidity equivalent to 3 parts of hydrogen per million parts of water.
e Qualitative only; acid reaction to methyl orange; high in chlorides, sulfates, calcium and magnesium; R_2O_3 present.

A few tests from other sources have been reported on the effect of sewage on the strength and time of setting of cement; many tests have been made using sea water. Earlier tests were limited in number and scope; they were in general made on cement or sand mortar briquets, hence gave little direct information on the concrete-making quality of the waters. This appears to be the first comprehensive study of the effect of impure mixing waters on the strength of *concrete*.

68 separate samples of water were tested in two different series of investigations. 52 samples were collected from many sections of the country. 14 of the samples were from the Chicago district. In general the waters were of a nature which was believed to be unsatisfactory for mixing concrete. Synthetic sea water and solutions of common salt were also used and parallel tests were made on several samples of clean, fresh waters. The samples may be classified as follows:

Fresh, clean waters (in general potable);
Minnesota bog waters;
Sea and alkali waters;
Mine and mineral waters;
Water contaminated with sewage and industrial wastes;
Solutions of common salt.

The tests described in this paper were confined to the use of impure waters in mixing concrete; the use of impure waters for curing concrete was not studied.

These tests were a part of the experimental studies of the properties of concrete and concrete materials being carried out through the co-operation of Lewis Institute and the Portland Cement Association at the Structural Materials Research Laboratory, Lewis Institute, Chicago.

OUTLINE OF TESTS.

This report includes two series of tests and gives the results of about 6000 strength tests of portland cement concrete and mortar. Series 137 was confined to a 1 : 4 concrete using 50 separate samples of water; in Series 138, 18 separate samples were used in concrete of a wide range of mixes and consistencies. In both series concrete tests were made at ages of 3 days to 2 1/3 yr.

Table 3.—Notes on Water Samples—Series 138.

Water samples (except Samples 101-105) were collected in 50-gallon lots in the Chicago District. Chemical analyses given in Table 4.

Sample	Remarks
101	Lake Michigan; Chicago city water supply.
102	Distilled water.
103	Distilled water-aerated. (Air, filtered through cotton, was passed through water for several hours).
	Synthetic sea water; 3.5 per cent solution of salts approximating composition of salts in sea water.
104	Medicine Lake, near Watertown, S. D.
105	Industrial sewage from Central Manufacturing District; collected at 39th and Morgan Sts.
106	Wash water from wood tanks used for beer storage; Atlas Brewing Co.
107	Chicago Drainage Canal at Ashland Ave. and 27th St.; contains sewage.
108	Waste water from Armour & Co., Soap Works, Chicago; contains some organic material.
109	Circulating water used to seal water gas machines at North Station, Peoples Gas Co.
110	Waste from plant of Corn Products Refining Co., Argo, Ill.
111	"Bubbly Creek;" waste from Chicago Stock Yards; very dark color; scum on surface.
112	Tan and dye liquor waste; from tannery of Griess, Pfleger & Co.
113	Soak water; waste from cleaning hides; from tannery of Griess, Pfleger & Co.
114	Lime soak from tannery used in removing hair from hides; from Illinois Sanitary District Testing
115	Station at tannery of Griess, Pfleger & Co.
116	Waste from settling tanks of Sherwin, Williams Co., paint manufacturers, Pullman, Ill.; contains sulphuric acid, organic matter, copper, and iron.
117	Spent acid plating bath, La Salle Steel Co., Cicero, Ill. Used in concrete diluted with 4 and 9 parts of water. Chemical analysis in Table 4 was made on the sample as received.

Series 137.—(Specimens made Aug. to Nov., 1920) consisted of compression tests of about 2200 4 x 8-in. concrete cylinders using 50 different samples of mixing water. A description of the waters is given in Table 1, chemical analyses in Table 2.

Tests of 1 : 4 concrete, relative consistency 1.10 were made for the following curing conditions: (Table 10)

(a) Moist room until tested at 3, 7 and 28 days, 3 mo., 1 and 2 1/3 yr.

(b) Moist room 28 days, remainder in air tested at 3 mo., 1 and 2 1/3 yr.

Time of setting and soundness tests of the cement using each mixing water are given in Table 8. Miscellaneous tests of cement and aggregates are given in Tables 5 to 7.

In general each value of concrete strength is the average of 5 or 6 tests made on different days.

Series 138.—(Specimens made April and May, 1921). This series included tests on 18 different waters most of which were from the Chicago district. Compression tests were made on about 2700 6 x 12-in. concrete cylinders, 540 (each) 2 x 4-in. standard-sand mortar cylinders and briquets. A description of the water samples is given in Table 3; chemical analyses in Table 4.

Tests were made as follows:

(a) Concrete, 1: 7, 1: 5, 1: 4, 1: 3 and 1: 2 mixes by volume; relative consistency 1.00; age at test 28 days. (Table 11.)

(b) Concrete, 1: 4 mix; relative consistency 0.90, 1.00, 1.10, 1.25 and 1.50; age at test 28 days. (Table 12.)

TABLE 4.—CHEMICAL ANALYSES OF WATER SAMPLES—SERIES 138.

Waters were from Chicago District, unless otherwise noted.
Analyses are expressed as parts per million of water as received.
See Table 3 for further details of origin and nature of samples

Mixing Water		Suspended Matter	Total Solids (105°C.)	Silica (SiO_2)	Iron and Aluminum Oxides (R_2O_3)	Calcium (Ca)	Magnesium (Mg)	Chloride (Cl)	Sulphate (SO_4)	Sulphide (S)	Oxygen to Oxidize Organic Matter
Sample	Source										
101	Lake Michigan	..	150	..	..	35	25	5	15	..	..
102	Distilled	..	...	..	..	..	..	...	...	..	..
103	Distilled aerated	..	...	..	..	..	..	...	...	..	..
104	Synthetic sea water*a*	..	35,000	..	..	420	1,310	19,400	2,700	..	..
105	Medicine Lake, S. D.*b*	55	53,100	..	..	420	5,750	540	28,700	..	..
106	Industrial sewage	10	310	10	5	50	180	45	0	0	5
107	Brewery waste	100	250	10	5	40	120	5	0	0	120
108	Drainage Canal	20	220	15	5	40	140	10	30	0	10
109	Soap works waste	20	410	5	5	60	190	90	60	0	15
110	Gas plant waste	120	470	30	5	40	240	170	150	35	420
111	Corn Products Co	170	980	15	5	90	370	330	120	5	75
112	Bubbly Creek	50	820	10	5	80	210	420	80	5	30
113	Tan and dye liquor (tannery)	110	1,840	25	5	70	150	480	560	20	310
114	Soak water (tannery)	90	3,700	20	5	85	200	2,340	60	10	35
115	Lime soak (tannery)	970	6,220*c*	30	0	990	trace	2,910	210	320	320
116	Paint manufacturing waste*d*	110	10,180	10	30	180	70	130	5,430	0	440
117	Spent plating bath*e*	90	...	..	..	0	0	810	114,400	..	..

a Sodium and potassium 78,600 parts per million; carbonate (CO_3) 70.
b Loss on ignition 8,610 parts per million; sodium (Na) 2820; potassium (K) 700; carbonate (CO_3) 430.
c Total solids after ignition.
d Loss on ignition 895 parts per million; copper, 170; zinc, trace.
e Ferrous iron 47,300, acidity (H_2SO_4) 31,250 parts per million. Diluted to 10 and 20 per cent of original concentration by the addition of Lake Michigan water before using in the tests.

(c) Concrete, 1: 4 mix; relative consistency 1.00; age at test 3, 7 and 28 days, 3 mo., 1 and 2 1/3 yr. (Table 13.)

(d) Briquet and 2 x 4-in. cylinder tests of 1: 3 standard sand at ages given under (c). (Table 14.)

(e) Time of setting and soundness tests of the cement were made using each mixing water. (Table 9.)

All concrete specimens were cured in moist room until test; mortar cylinders and briquets were cured in water after the first 24 hr. and tested at the ages given above.

The strengths of concrete given in the tables are the average of 10 tests made on different days; for the mortar, 5 tests.

MATERIALS.

Water.—39 of the 50 samples of water in Series 137 were collected in about 10-gal. lots. About 50-gal. lots of 13 different waters representing what were considered the worst qualities to be found in the Chicago district were collected for test in Series 138. The solutions of common salt and the synthetic sea waters were prepared at the Laboratory; the distilled waters were purchased in Chicago; the Lake Michigan waters were from the city water supply. For details of the water samples see Tables 1 to 4.

TABLE 5.—CHEMICAL ANALYSIS OF CEMENTS.

The percentages are the average of 2 determinations.

Cement	Used in Series	Silica (SiO_2)	Iron and Aluminum Oxides ($Fe_2O_3 + Al_2O_3$)	Calcium Oxide (CaO)	Magnesium Oxide (MgO)	Sulfuric Anhydride (SO_3)
4951..............	137	21.88	9.46	62.64	2.94	1.55
5075..............	137	21.73	9.15	62.90	2.79	1.62
5259..............	138	21.85	9.25	61.97	2.78	1.68

Portland Cement consisted of a mixture of equal parts of 5 brands purchased in Chicago. About the first two-thirds of the specimens in Series 137 were made from Cement Lot 4951, and the remainder from Cement 5075. In Series 138, concrete and mortar tests were made from Cement 5259. Chemical analyses of the cements are given in Table 5; standard-sand mortar tests of the cements using Lake Michigan water are given in Table 6.

Aggregate for concrete tests in both series consisted of sand and pebbles from Elgin, Ill. For Series 137 the aggregate was graded 0 to ¾-in.; for Series 138, 0 to 1½-in. In order to secure a uniform grading of aggregate, the sand and pebbles were separated into the following sizes and recombined to the sieve analyses shown in Table 7: 0-No. 4; No. 4-⅜ in.; ⅜-¾; ¾-1½.

TEST PIECES.

In Series 137 the strength tests were made on 4 x 8-in. concrete cylinders; in Series 138, on 6 x 12-in. cylinders. In Series 138 compression tests were made on 2 x 4-in. cylinders and tension tests on briquets of 1 : 3 standard sand. Concrete specimens were made, cured and tested in accordance with "Tentative Methods of Making Compression Tests of Concrete" (Serial Designation: C39-21T) of the American Society for Testing

Materials; mortar specimens in accordance with the Standard and Tentative Specifications of the American Society for Testing Materials.

The concrete mixtures were expressed in terms of 1 volume of cement to a given number of volumes of aggregate, *dry and rodded* after combining to the grading used. Concrete was mixed by hand with a blunted trowel in metal pans in batches of sufficient size for three 4 x 8-in. cylinders or one 6 x 12-in. cylinder.

The quantity of mixing water was the same for similar concrete, except insofar as differences in normal consistency of the cement dictated small variations. The plasticity of the concrete was measured by means of the "flow-table".* The flow-table consists of a circular metal plate attached to a frame in such a way that it can be raised and dropped by means of a

TABLE 6.—MORTAR TESTS OF CEMENTS.

1:3 standard sand mortar.
Portland cements a mixture of equal parts of 5 brands purchased in Chicago.
Mixing water from Chicago city water supply.
Tests made in accordance with Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials.
Each value for strength is the average of 5 tests made on different days.
Time of setting tests given in Tables 8 and 9.

Cement	Used in Series	Fineness Residue on 200 Sieve	Normal Consistency	Tensile Strength of Briquets, lb. per sq. in.						Compression Tests of 2x4-in. Cylinders lb. per sq. in.					
				7 days	28 days	3 mos.	6 mos.	1 year	2 years	7 days	28 days	3 mos.	6 mos.	1 year	2 years
4951..	137	18.2	22.5	270	360	425	390	400	375	2220	3740	4340	4840	4260	4800
5075..	137	18.8	22.5	250	350	420	445	340	380	1710	3010	4020	3640	4030	3630
5259..	138	18.2	24.0	255	375	410	...	415	395*	1700	2800	3550	....	3730	3840*

* Age of test, 2⅓ years.

cam. The test is made by rodding the freshly-mixed concrete into the mold on the metal table. The mold consists of a truncated metal cone (top diameter 6¾ in., bottom diameter 10 in., height 5 in.). After the mold is filled, it is immediately withdrawn and the table raised and dropped ½ in., 15 times in about 10 seconds. The base diameter of the mass of concrete after the test, expressed as a percentage of the original diameter, is the "flow."

The concrete cylinder forms were of cold-drawn steel tubing, slotted along one element. In making the specimens the forms were placed on machined cast-iron plates. The concrete was placed in 3 layers, each layer being rodded 30 times with a ⅝-in. bullet-pointed steel rod. Three to four hours after making, the specimens were capped with neat cement

*See "Measuring Flowability of Concrete by Flow Table," Concrete, June, 1920, v. 16, p. 274; Eng. News-Rec., May 27, 1920, p. 1044.

Time of Set of Concrete, by Watson Davis; Proc. Am. Soc. Testing Materials, v. 21, p. 995, 1921.

Economic Value of Admixtures, by Pearson and Hitchcock; Proc. Am. Concrete Inst., 1924.

(which had been allowed to stand 3 to 6 hr. after mixing) and the cap made plane by means of a machined plate which was worked down to the top of the form.

The forms were removed after 16 to 20 hr. and the specimens placed in the moist room. In general they remained in the moist room until test; for a portion of Series 137 they were cured in air after 28 days in the moist room.

Concrete tests were made in a 200,000-lb. universal testing machine. The load was applied through a spherical bearing block placed on top of the cylinder.

TABLE 7.—SIEVE ANALYSIS OF AGGREGATES.

Sieve analyses made using square mesh wire cloth manufactured by The W. S. Tyler Co., Cleveland.
Aggregate: Sand and pebbles from Elgin, Ill.; largely limestone origin. Sand contained 2 per cent silt, mostly clay.

Series	Kind of Aggregate	Amounts Coarser than Each Sieve, per cent by weight									Fineness Modulus*	Unit Weight, lb. per cu. ft.
		No. 100	No. 48	No. 28	No. 14	No. 8	No. 4	⅜ in.	¾ in.	1½ in.		
137	Sand	99	88	55	38	20	0	..	..	..	3.00	112
	Pebbles	100	100	100	100	100	100	67	0	..	6.67	110
	Mixed concrete aggregate	99	94	79	71	63	55	39	0	..	5.00	126
138	Sand	98	87	58	38	16	3	0	..	..	3.00	112
	Pebbles	100	100	100	100	100	100	75	25	0	7.00	110
	Mixed concrete aggregate	99	96	86	79	72	68	50	17	0	5.67	128

* Fineness modulus of aggregate is the sum of the percentages in the sieve analysis, divided by 100.

DISCUSSION OF TESTS.

The results of the strength tests of concrete and mortar are given in Tables 10 to 22 and Figs. 1 to 7. The most striking feature of this investigation was that in spite of the wide range in type of mixing waters, the samples, with a few exceptions, gave good results in concrete.

The waters used ranged from distilled water to water containing nearly 250,000 parts per million of solids. The impurities consisted of alkaline salts, sewage, industrial wastes of various kinds and miscellaneous salts, minerals and other impurities found in mines, deep wells, bogs, etc.

The following discussion points out briefly the principal features of these tests. Chief emphasis is placed on the results of tests of the 1 : 4 concrete; this mix was used throughout Series 137 and in the bulk of the tests in Series 138 and represents the quality of concrete used in road construction and other high-grade work.

Criterion of Quality of Mixing Water.—In this discussion the quality of mixing water will be measured by comparing its concrete strength with that of fresh water tested at the same time, in the same mix, at the same age, etc. The "strength-ratios" computed on this basis are given in the tables. While the lowest permissible strength-ratio is a matter of judg-

TABLE 8.—TIME OF SETTING OF CEMENT AS AFFECTED BY IMPURE WATERS—SERIES 137.

Portland Cement consisted of a mixture of equal parts of 5 brands purchased in Chicago. (Cement 4,951, see Table 6.)

Tests were made in accordance with Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials.

Each value is the average of 2 tests made on different days.

Group	Classification	Sample	Source of Mixing Water	Normal Consistency	Vicat Needle Initial h.	m.	Vicat Needle Final h.	m.	Gillmore Needle Initial h.	m.	Gillmore Needle Final h.	m.	Soundness (over boiling water)
1	Fresh waters	2, 50	Lake Michigan	22.5	3	25	6	30	4	30	6	30	OK
		1, 49	Distilled water	22.5	3	40	7	00	4	30	7	00	OK
		14	City water, Coal City, Ill.	22.5	3	40	6	30	4	30	6	30	OK
		15	City water, Tracy, Minn.	23.0	4	00	8	00	4	30	8	00	OK
		17	City water, Marshall, Minn.	22.5	4	00	6	20	5	00	7	10	OK
		18	City water, Dundee, Mich.	22.5	3	40	6	50	5	00	7	25	OK
		36	Lake County, Ind.	22.5	4	30	7	30	4	30	7	30	OK
		37	Youghiogheny River	22.5	4	00	8	00	5	15	8	15	OK
			Average	....	3	52	7	20	4	43	7	18	
2	Minnesota bog waters	19	Pale Face River	22.5	3	45	6	30	5	00	7	15	OK
		20	Helwig Creek	22.5	3	30	6	30	4	45	7	15	OK
		21	Cloquet River	22.5	3	50	6	40	5	00	7	00	OK
		24	Dunka River	23.0	3	45	8	00	4	45	8	00	OK
		22	Stagnant pool	22.5	3	30	6	45	4	30	7	10	OK
		23	Sulphur Dam	22.5	3	30	7	00	4	30	7	30	OK
		25	Sulphur Dam	22.5	3	30	6	10	4	30	6	45	OK
			Average	....	3	37	6	48	4	43	7	16	
3	Sea water and alkali waters	3	Sea water	22.5	3	45	7	15	4	30	7	45	OK
		16	Sea water	22.5	2	45	6	00	4	00	6	30	OK
		4	Synthetic sea water	22.5	3	30	7	00	4	35	7	00	OK
		5	Great Salt Lake	24.0	5	45*	11	15*	7	30*	11	15*	OK
		26	Devil's Lake N. D.	23.0	3	30	7	00	4	45	7	00	OK
		27	Tile Drain, Minn.	22.5	3	30	7	00	5	00	7	00	OK
		28	Tile Drain, N. M.	22.5	3	15	6	00	4	30	6	50	OK
		29	Tile Drain, Colo.	22.5	4	00	6	15	4	40	6	45	OK
		31	Muddy Creek, Utah	23.0	3	15	8	00	4	45	8	00	OK
		36	Price, Utah	23.0	3	15	7	00	4	20	7	30	OK
			Average	....	3	25	6	50	4	36	7	10	
4	Solutions of common salt	6	1 per cent common salt	22.5	3	30	7	00	4	35	7	15	OK
		7	2 per cent common salt	22.5	3	30	7	30	4	45	7	30	OK
		8	5 per cent common salt	23.0	3	45	7	30	4	45	7	30	OK
		9	10 per cent common salt	24.0	4	25	9	30	5	45	9	45	OK
		10	15 per cent common salt	24.5	5	00	9	30	6	30	9	45	OK
		11	20 per cent common salt	25.0	6	00	10	30	9	30	......		
			Average	....	4	22	8	35	5	58	8	21	
5	Mine and mineral waters	12	Mineral water, Colo.	23.0	3	30	6	55	5	00	7	00	OK
		30	Mine water, Ariz.	23.0	4	00	6	25	5	00	6	50	OK
		32	Loop Creek, W. Va.	22.5	3	45	7	45	4	45	7	45	OK
		33	Coal mine, Ill.	23.0	3	45	7	45	4	40	7	50	OK
		34	Coal mine, Ill.	23.5	3	45	8	00	5	00	8	15	OK
		35	Gypsum mine, Ia.	23.0	3	45	8	00	5	00	8	15	OK
		43	Coal washery, Pa.	23.0	4	00	7	30	4	45	8	00	OK
			Average	....	3	47	7	29	4	53	7	42	
6	Waters containing industrial wastes and sewage	13	Illinois River	23.0	4	00	6	50	5	00	7	15	OK
		38	Lakehurst Air Station	22.5	4	00	7	30	4	30	7	45	OK
		39	Monongahela River	22.5	3	45	7	30	4	30	8	00	OK
		40	Refuse, Oil Refinery	26.0	11	00*	36	00*	9	00*	48	00*	OK
		42	Endicott, N. Y. (tannery)	22.0	3	30	7	45	4	50	8	00	OK
		44	"Bubbly Creek," Chicago	22.5	4	00	7	30	4	45	7	50	OK
		45	Cygnet, Ohio	23.0	3	15	7	00	4	20	7	45	OK
		47	Drainage Canal, Chicago	22.5	3	35	7	15	4	45	8	00	OK
		48	Allegheny River (low water)	22.5	3	45	7	15	4	40	7	45	OK
		41	Allegheny River (high water)	22.5	3	45	7	45	4	45	7	45	OK
			Average	....	3	44	7	22	4	41	7	47	

* Omitted from average.

ment, we have considered that waters which give strength-ratios less than 85 per cent were unsatisfactory for mixing concrete. This is a rigid requirement, since some allowance must be made for accidental variations in the tests. Such a rigid requirement is justified in this discussion only because we have made a large number of tests at the same time, under carefully controlled conditions. Where isolated tests of concrete or mortar are relied upon, the limiting strength-ratio should be reduced to 80 per cent or 75 per cent.

TABLE 9.—TIME OF SETTING OF CEMENT—SERIES 138.

Cement: a mixture of 5 brands of Portland Cement purchased in Chicago, see Tables 5 and 6 for chemical analysis and physical tests.

Tests made in accordance with Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials.

Each value is the average of 2 tests made on different days.

Mixing Water		Normal Consistency	Time of Setting								Soundness (over boiling water)
			Vicat Needle				Gillmore Needle				
			Initial		Final		Initial		Final		
Sample	Source		h.	m.	h.	m.	h.	m.	h.	m.	
101	Lake Michigan	24.0	4	25	8	40	6	40	10	35	OK
102	Distilled	23.5	3	30	8	10	6	10	9	20	OK
103	Distilled aerated	24.0	3	45	8	45	6	25	9	20	OK
	Average	24.0	3	55	8	30	6	25	9	45	
104	Synthetic sea water	24.0	2	30	6	20	5	15	8	25	OK
105	Medicine Lake, S. D.	25.0	4	20	9	00	6	40	10	50	OK
106	Industrial sewage	24.0	5	00	9	15	7	30	11	50	OK
107	Brewery waste	23.5	4	00	8	50	6	30	10	30	OK
108	Drainage Canal	24.0	4	30	9	00	6	40	10	30	OK
109	Soap works waste	24.0	4	20	8	40	7	10	10	55	OK
110	Gas plant waste	24.0	5	05	9	05	7	00	10	50	OK
111	Corn Products Co.	24.5	5	30	9	20	7	50	11	15	OK
112	"Bubbly Creek"	24.0	4	25	8	50	7	10	10	35	OK
113	Tan and dye liquor (tannery)	24.0	4	50	8	50	7	10	10	30	OK
114	Soak water	24.0	3	50	8	40	6	20	10	40	OK
115	Lime soak (tannery)	23.5	4	00	8	00	7	15	11	10	OK
116	Paint manufacturing waste	23.5	3	30	8	10	6	35	10	50	OK
117 (10)*	Spent plating bath	26.0	4	50	7	55	7	15	12	20	OK
117 (20)*	Spent plating bath	27.0	5	45	11	15	8	05	13	00	OK

* Percentage of Sample 117 mixed with Lake Michigan water.

The following waters gave concrete strength-ratios below 85 per cent, hence are considered unsatisfactory:

Acid waters,
Lime soak from tannery,
Refuse from paint factory,
Waters containing over 5 per cent of common salt,
Mineral water from Montrose, Colorado.

Fresh waters were collected from city supplies and rivers and were such as would usually be accepted without question for mixing concrete.

Table 10.—Compression Tests of Concrete Using Impure Mixing Waters—Series 137.

4 by 8-in. concrete cylinders.
Mix 1–4 by volume.
Relative consistency 1.10; water-ratio 0.88; plasticity measured by means of flow table.
Aggregate: sand and pebbles from Elgin, Ill., graded 0-¾-in.
Each value is the average of 5 or 6 tests made on different days.
The values in parentheses are "Strength Ratios," that is percentages of the average strength in Group 1 or a given age and curing condition.
See Figs. 1 to 3.

Sample	Source of Mixing Water	Flow	Compressive Strength, lb. per sq. in.								
			Cured in Moist Room Until Test						Cured in Moist Room 28 Days, Remainder in Air		
			3 days	7 days	28 days	3 mo.	1 year	2⅓ years	3 mo.	1 year	2⅓ years
	Group 1.—Fresh Waters										
2	Lake Michigan...........	212	960 (100)	1510 (94)	3160 (102)	4610 (106)	5650 (104)	5770 (112)	4730 (109)	4060 (104)	3620 (108)
50	Lake Michigan...........	229	870 (91)	1430 (89)	3120 (101)	4000 (92)	5320 (98)	5750 (112)	4130 (95)	3970 (102)	2670 (80)
1	Distilled water...........	214	900 (94)	1630 (102)	3020 (98)	4560 (105)	5430 (100)	5250 (102)	4330 (99)	3750 (96)	3390 (102)
49	Distilled water...........	218	950 (99)	1560 (98)	2780 (90)	3480 (80)	5390 (99)	3540 (69)	4400 (101)	3430 (88)	3320 (99)
14	City water, Coal City, Ill.	216	990 (103)	1560 (98)	2870 (93)	4150 (96)	5140 (94)	5070 (99)	3800 (87)	3980 (102)	2790 (84)
15	Tracy, Minn.............	224	900 (94)	1550 (97)	2760 (90)	4160 (96)	5360 (98)	4770 (93)	4120 (95)	3790 (97)	3400 (102)
17	Marshall, Minn..........	216	1020 (106)	1760 (110)	3140 (102)	4730 (109)	5740 (105)	5740 (112)	4540 (104)	3960 (102)	3710 (111)
18	Dundee, Mich...........	214	1040 (108)	1700 (106)	3140 (102)	4690 (108)	5440 (100)	5640 (110)	4780 (110)	3940 (101)	3600 (108)
36	Lake County, Ind........	214	990 (103)	1620 (101)	3450 (112)	4610 (106)	5480 (101)	4340 (85)	4580 (105)	4120 (106)	3380 (101)
37	Youghiogheny River.....	214	930 (97)	1650 (103)	3400 (110)	4390 (101)	5540 (102)	5390 (105)	4100 (94)	3960 (102)	3540 (106)
	Average.............	216	960 (100)	1600 (100)	3080 (100)	4340 (100)	5450 (100)	5130 (100)	4350 (100)	3900 (100)	3340 (100)
	Group 2.—Minnesota Bog Waters										
19	Pale Face River.........	220	830 (86)	1720 (107)	3410 (111)	4860 (112)	5980 (110)	5000 (98)	4380 (101)	4290 (110)	3070 (92)
20	Helwig Creek............	217	830 (86)	1740 (108)	3070 (100)	4390 (101)	5430 (100)	4220 (82)	..	..	..
21	Cloquet River...........	217	960 (100)	1780 (111)	2750 (89)	4160 (96)	4920 (90)	4700 (92)	..	..	..
24	Dunka River............	221	930 (97)	1750 (109)	3130 (102)	..	..	..	..	..	..
22	Stagnant pool............	218	1000 (104)	1680 (105)	3410 (111)	4350 (98)	5100 (94)	5390 (105)	..	..	..
23	Sulphur Dam............	220	1080 (112)	1850 (116)	3280 (106)	4180 (96)	5870 (108)	4350 (85)	..	..	..
25	Sulphur Dam............	217	880 (92)	1550 (97)	2970 (96)	4710 (109)	5540 (102)	5840 (114)	..	..	..
	Average.............	219	930 (97)	1720 (107)	3150 (102)	4440 (102)	5470 (100)	4920 (96)			

TABLE 10.—COMPRESSION TESTS OF CONCRETE USING IMPURE MIXING WATERS—SERIES 137 (*continued*).

Sample	Source of Mixing Water	Flow	Compressive Strength, lb. per sq. in.								
			Cured in Moist Room Until Test						Cured in Moist Room 28 Days, Remainder in Air		
			3 days	7 days	28 days	3 mo.	1 year	2½ years	3 mo.	1 year	2½ years
	GROUP 3.—SEA WATER AND ALKALI WATERS										
3	Sea water	209	1200 (125)	1700 (106)	2790 (90)	3470 (80)	4840 (89)	4230 (82)	4030 (93)	3880 (99)	3460 (104)
16	Sea water	220	1180 (123)	1840 (115)	2650 (86)	3490 (81)	4600 (84)	4510 (88)	3600 (83)	3910 (100)	2980 (89)
4	Synthetic sea water	214	1320 (137)	1490 (93)	2610 (85)	3640 (84)	4520 (83)	4520 (88)	3740 (86)	3750 (96)	3660 (110)
5	Great Salt Lake	200	1080 (112)	1460 (91)	2380 (77)	3300 (76)	3870 (71)	3360 (65)	3800 (87)	4090 (105)	3260 (98)
26	Devil's Lake, N. D.	221	980 (102)	1730 (107)	3050 (99)	4180 (96)	5360 (98)	4620 (90)	4140 (95)	3560 (91)	3340 (100)
27	Tile drain, Minn.	216	1020 (106)	1720 (107)	3310 (108)	4650 (107)	5680 (104)	4690 (91)	4350 (100)	4240 (109)	3970 (119)
28	Tile drain, N. Mex.	210	1210 (126)	1800 (112)	3160 (102)	4170 (96)	5420 (100)	5070 (99)	4620 (106)	3780 (97)	3590 (107)
29	Tile drain, Colo.	213	1030 (107)	1730 (107)	3180 (103)	4340 (100)	5710 (105)	4910 (96)	4320 (99)	3910 (100)	3320 (99)
31	Muddy Creek, Utah	219	1180 (123)	1480 (93)	2790 (90)	3880 (89)	5340 (98)	5010 (98)	4540 (104)	3660 (94)	3390 (102)
46	Price, Utah	233	1090 (113)	1490 (93)	2820 (91)	4190 (97)	4990 (92)	4450 (87)	4290 (99)	3500 (90)	3110 (93)
	Average	216	1130 (118)	1640 (102)	2870 (92)	3930 (91)	5030 (92)	4540 (88)	4140 (95)	3830 (98)	3410 (102)
	GROUP 4.—SOLUTIONS OF COMMON SALT										
6	1 per cent common salt	212	1070 (111)	1680 (105)	2600 (84)	3720 (86)	4710 (86)	4490 (87)	4030 (93)	3420 (88)	3700 (111)
7	2 per cent common salt	221	1210 (126)	1640 (102)	2710 (88)	3830 (88)	4660 (86)	4840 (94)	3640 (84)	3630 (93)	3240 (97)
8	5 per cent common salt	234	1120 (117)	1450 (90)	2310 (75)	2900 (67)	3940 (72)	4050 (79)	3360 (77)	3690 (95)	3670 (110)
9	10 per cent common salt	238	1030 (107)	1400 (87)	2100 (68)	2750 (63)	3650 (67)	3050 (59)	2850 (66)	3520 (90)	3250 (97)
10	15 per cent common salt	240	850 (89)	1200 (75)	2020 (65)	2840 (65)	3080 (57)	2880 (56)	2760 (63)	3640 (93)	3640 (109)
11	20 per cent common salt	238	740 (77)	1130 (70)	1870 (61)	2660 (61)	3210 (59)	3150 (61)	2950 (68)	3450 (88)	3960 (119)
	Average	230	1000 (104)	1420 (89)	2270 (73)	3120 (72)	3880 (71)	3740 (73)	3260 (75)	3560 (91)	3530 (106)

TABLE 10.—COMPRESSION TESTS OF CONCRETE USING IMPURE MIXING WATERS—SERIES 137 (*continued*).

Sample	Source of Mixing Water	Flow	Compressive Strength, lb. per sq. in.								
			Cured in Moist Room Until Test						Cured in Moist Room 28 Days, Remainder in Air		
			3 days	7 days	28 days	3 mo.	1 year	2½ years	3 mo.	1 year	2½ years
	GROUP 5.—MINE AND MINERAL WATERS										
12	Mineral water, Colo......	220	1020 (106)	1600 (100)	2360 (77)	3590 (83)	4650 (85)	4780 (93)	3710 (85)	3240 (83)	2660 (80)
30	Mine waters, Ariz........	219	870 (91)	1610 (101)	3090 (100)	..	..	..	..	..	..
32	Loop Creek, W. Va......	213	960 (100)	1670 (104)	3150 (102)	4440 (102)	5480 (101)	4500 (88)	4260 (98)	3810 (98)	3420 (102)
33	Coal mine, Ill............	219	1010 (105)	1630 (102)	3150 (102)	4020 (93)	5580 (102)	5260 (103)	4420 (102)	3630 (93)	3660 (110)
34	Coal mine, Ill...........	215	1110 (116)	1660 (104)	3140 (102)	4420 (102)	5450 (100)	4080 (80)	4310 (99)	3390 (87)	3800 (114)
35	Gypsum mine, Ia........	222	1050 (109)	1700 (106)	3050 (99)	4560 (105)	5660 (104)	4690 (91)	4760 (110)	4040 (104)	3280 (98)
43	Coal washery, Pa........	230	1010 (105)	1600 (100)	2990 (97)	4470 (103)	5640 (104)	4780 (93)	3790 (87)	3720 (95)	3490 (105)
	Average.............	220	1000 (104)	1640 (102)	2990 (97)	4250 (98)	5410 (100)	4680 (91)	4210 (97)	3640 (93)	3390 (102)
	GROUP 6.—WATERS CONTAINING INDUSTRIAL WASTE AND SEWAGE										
13	Illinois River............	224	970 (101)	1490 (93)	2570 (83)	3690 (85)	4920 (90)	4650 (91)	3990 (92)	3590 (92)	3390 (102)
38	Lakehurst Air Station....	214	880 (92)	1570 (98)	3050 (99)	4420 (102)	5560 (102)	4060 (79)	4440 (101)	3870 (99)	2960 (89)
39	Monongahela River.....	217	940 (98)	1530 (96)	3070 (100)	4280 (99)	5360 (98)	4540 (88)	..	..	..
40	Refuse, oil refinery.......	182	630 (66)	1840 (115)	2890 (94)	3820 (88)	4510 (83)	4880 (95)	..	..	..
42	Endicott, N. Y. (tannery)	226	950 (99)	1370 (86)	2510 (81)	3740 (86)	4590 (84)	4190 (82)	3410 (78)	3480 (89)	2790 (84)
44	"Bubbly Creek," Chicago	226	1090 (114)	1540 (96)	2950 (96)	4330 (100)	5500 (101)	4790 (94)	4400 (101)	3540 (91)	3360 (100)
45	Cygnet, Ohio............	224	1100 (115)	1820 (114)	2660 (86)	3890 (90)	4960 (91)	3950 (77)	4250 (98)	3790 (97)	2930 (88)
47	Drainage Canal..........	217	990 (103)	1540 (96)	3000 (97)	4210 (97)	5120 (94)	4810 (94)	4570 (105)	3740 (96)	3910 (117)
48	Allegheny River (low water)...............	219	940 (88)	1660 (104)	3030 (98)	4560 (105)	5660 (104)	4850 (95)	4290 (99)	4040 (104)	3500 (105)
41	Allegheny River (high water)................	222	900 (94)	1500 (94)	3000 (97)	4350 (100)	5730 (105)	3820 (74)	..	..	..
	Average.............	217	940 (98)	1590 (99)	2870 (93)	4130 (95)	5190 (95)	4450 (87)	4190 (96)	3730 (95)	3260 (98)

TABLE 11.—TESTS OF CONCRETE OF DIFFERENT MIXTURES—SERIES 138.

Compression tests of 6 x 12-in. concrete cylinders.
Mix by volume; relative consistency 1.00.
Aggregate: Sand and pebbles from Elgin, Ill., graded 0 to 1½ in.
Age at test, 28 days.
Each value is the average of 10 tests made on different days unless otherwise noted.
Values for compressive strength expressed in pounds per square inch.
The values in parentheses are "Strength Ratios," that is, percentages of the average strength for fresh waters.
See Fig. 4.

Mixing Water		1:7		1:5		1:4*a*		1:3		1:2		Average	
Sample	Source	Water-Ratio *b* 1.03		Water-Ratio *b* 0.85		Water-Ratio *b* 0.75		Water-Ratio *b* 0.65		Water-Ratio *b* 0.55		Water-Ratio *b* 0.77	
		Flow	Compressive Str.	Flow	Compressive Str.	Flow	Compressive Str.	Flow	Compressive Str.	Flow	Compressive Str.	Flow	Compressive Str.
101	Lake Michigan	151	1880	178	3010	181	3670	187	4480	187	5350	177	3680
102	Distilled	144	1890	158	3080	171	3680	176	4440	174	5060	163	3630
103	Distilled aerated	142	1900	156	3070	178	3700	173	4600	171	5100	162	3670
	Average	146	1890 (100)	164	3050 (100)	177	3680 (100)	179	4510 (100)	177	5170 (100)	167	3660 (100)
104	Synthetic sea water	145	1840 (97)	155	2820 (93)	177	3290 (89)	171	3980 (88)	173	4670 (90)	162	3310 (91)
106	Industrial sewage	154	1850 (98)	168	3150 (103)	184	3780 (103)	169	4480 (99)	182	5220 (101)	171	3680 (100)
107	Brewery waste	157	1980 (106)	178	3120 (102)	194	3800 (104)	180	4500 (100)	196	5230 (101)	181	3730 (102)
108	Drainage Canal	154	2150 (114)	179	3230 (106)	191	3870 (105)	190	4390 (97)	194	5300 (102)	180	3790 (103)
109	Soap works waste	162	1980 (105)	176	3110 (102)	193	3770 (103)	190	4400 (98)	190	5210 (101)	181	3690 (101)
110	Gas plant waste	159	1870 (99)	177	3050 (100)	195	3630 (99)	186	4290 (95)	197	4880 (94)	182	3540 (97)
111	Corn Products Co.	156	1840 (97)	163	3050 (100)	184	3460 (94)	184	4240 (94)	179	4930 (95)	171	3500 (96)
112	"BubblyCreek," Chicago	150	1980 (105)	176	3030 (99)	192	3720 (101)	181	4370 (97)	191	5050 (97)	177	3630 (99)
113	Tan and dye liquor (tannery)	152	1880 (99)	172	2990 (98)	185	3630 (99)	189	4440 (98)	189	4950 (96)	176	3680 (98)
114	Soak water (tannery)	157	1840 (97)	170	2870 (94)	187	3480 (94)	187	4160 (92)	184	4860 (94)	177	3440 (94)
115	Lime soak (tannery)	178	1580 (83)	200	2450 (80)	211	3130 (85)	212	3720 (82)	203	4160 (80)	201	3010 (82)
116	Paint manufacturing waste	158	1660 (88)	167	2730 (89)	190	3210 (87)	190	3580 (79)	187	4620 (89)	177	3160 (86)
117 (10) *c*	Spent plating bath	139	2270 (120)	131	3330 (109)	148	3500 (95)	141	4170 (93)	145	4600 (89)	139	3570 (98)
117 (20) *c*	Spent plating bath	139	2040 (108)	124	3020 (99)	136	3250 (88)	149	3770 (83)	153	4350 (84)	140	3290 (90)
	Average of impure waters	154	1910 (101)	168	2990 (98)	183	3540 (96)	180	4170 (92)	183	4870 (95)	172	3490 (95)

a Values are average of 30 tests from 3 groups; same values are given in Tables 12 and 13.
b 10% more water was used for Sample 117 (20) due to quick stiffening of concrete.
c Percentage of Sample 117 used with Lake Michigan water.

TABLE 12.—TESTS OF CONCRETE OF DIFFERENT CONSISTENCIES—SERIES 138.

Compression tests of 6 x 12-in. cylinders.
Mix 1:4 by volume.
Aggregate: Sand and pebbles from Elgin, Ill., graded 0 to 1½ in.
Age at test, 28 days.
Each value is the average of 10 tests made on different days unless otherwise noted.
The values for strength are expressed in pounds per square inch.
The values in parentheses are "Strength Ratios," that is, percentages of the average strength for fresh waters.
See Fig. 5.

Mixing Water		0:90		1:00a		1:10		1:25		1:50		Average	
Sample	Source	Water-Ratio c 0.68		Water-Ratio c 0.75		Water-Ratio c 0.82		Water-Ratio c 0.92		Water-Ratio c 1.09		Water-Ratio c 0.85	
		Flow	Compressive Str.	Flow	Compressive Str.	Flow	Compressive Str.	Flow	Compressive Str.	Flow	Compressive Str.	Flow b	Compressive Str.
101	Lake Michigan.........	130	4300	181	3670	215	3350	d	2850	d	1540	172	3140
102	Distilled...............	131	4170	171	3680	219	3270	...	2470	...	1620	177	3040
103	Distilled aerated.......	135	4140	178	3700	216	3420	...	2430	...	1600	177	3060
	Average...........	132	4200 (100)	177	3680 (100)	217	3350 (100)	...	2580 (100)	...	1590 (100)	174	3080 (100)
104	Synthetic sea water....	136	3660 (87)	177	3290 (89)	205	2830 (85)	...	2240 (87)	...	1470 (92)	174	2690 (87)
106	Industrial sewage.......	136	4110 (98)	184	3780 (103)	233	3230 (96)	...	2450 (95)	...	1580 (100)	183	3030 (98)
107	Brewery waste.........	142	4250 (101)	194	3800 (104)	234	3360 (100)	...	2570 (100)	...	1580 (100)	190	3110 (101)
108	Drainage Canal........	144	4380 (104)	191	3870 (105)	219	3470 (104)	...	2670 (104)	...	1530 (96)	184	3180 (103)
109	Soap works waste.......	141	4230 (101)	193	3770 (103)	206	3330 (99)	...	2500 (97)	...	1550 (97)	180	3080 (100)
110	Gas plant waste........	141	4000 (95)	195	3630 (99)	226	3210 (96)	...	2380 (92)	...	1520 (96)	183	2950 (96)
111	Corn Products Co......	136	3830 (91)	184	3460 (94)	226	3200 (96)	...	2510 (97)	...	1480 (93)	182	2900 (94)
112	"Bubbly Creek," Chicago	141	4000 (95)	192	3720 (101)	228	3300 (99)	...	2500 (97)	...	1580 (100)	187	3020 (98)
113	Tan and dye liquor (tannery)...............	133	4060 (97)	185	3630 (99)	220	3100 (93)	...	2260 (88)	...	1500 (94)	178	2910 (95)
114	Soak water (tannery)...	137	3890 (93)	187	3480 (94)	225	3900 (92)	...	2390 (93)	...	1370 (86)	183	2840 (92)
115	Lime soak (tannery)....	158	3500 (83)	211	3130 (85)	243	2640 (79)	...	2030 (79)	...	1260 (79)	202	2510 (82)
116	Paint manufacturing waste...............	141	3740 (89)	190	3210 (87)	233	2850 (85)	...	2150 (83)	...	1340 (84)	188	2660 (86)
117 (10) e	Spent plating bath.....	118	3790 (90)	148	3500 (95)	169	3140 (94)	...	2510 (97)	...	1680 (106)	143	2920 (95)
117 (20) e	Spent plating bath.....	115	3910 (93)	136	3250 (88)	162	3160 (94)	...	2500 (97)	...	1650 (104)	136	2890 (94)
	Average of impure waters.........	137	3950 (94)	183	3540 (96)	216	3130 (93)	...	2400 (93)	...	1570 (99)	178	2900 (94)

a Values are average of 30 tests from 3 groups. Same values are given in Tables 11 and 13.
b Flows average of relative consistencies, 0.90, 1.00, and 1.10 only.
c 10% more mixing water was used for Sample 117 (20) due to quick stiffening.
d Over limit of flow table.
e Percentage of Sample 117 used with Lake Michigan water

TABLE 13.—TESTS OF CONCRETE AT DIFFERENT AGES—SERIES 138.

Compression tests of 6 x 12-in. concrete cylinders.
Mix 1:4 by volume.
Relative consistency 1.00; water-ratio *a* 0.75.
Aggregate sand and pebbles from Elgin, Ill., graded 0 to 1½-in.
Values for compressive strength expressed in pounds per square inch.
Each value is the average of 10 tests made on different days unless otherwise noted.
The values in parenthesis are "Strength Ratios," that is, percentages of the average strength for fresh waters.
See Fig. 6.

Mixing Water		Flow	Compressive Strength, lb. per sq. in.					
Sample	Source		3 days	7 days	28 days *b*	3 months	1 year	2⅓ years
101	Lake Michigan	181	1240	2150	3670	5350	6140	6200
102	Distilled water	171	1270	2240	3680	5340	6060	6680
103	Aerated distilled	178	1170	1990	3700	5140	5810	6500
	Average	177	1230 (100)	2130 (100)	3680 (100)	5280 (100)	6000 (100)	6460 (100)
104	Synthetic sea water	177	1590 (129)	2250 (106)	3290 (89)	4200 (79)	5260 (88)	5380 (81)
105	Medicine Lake, S. D.	141	1350 (110)	2170 (102)	3220 (88)	4430 (84)	5270 (88)	5950 (92)
106	Industrial sewage	184	1230 (100)	2090 (98)	3780 (103)	5220 (99)	5910 (99)	6420 (99)
107	Brewery waste	194	1190 (97)	2080 (98)	3800 (104)	5290 (100)	5710 (95)	6060 (94)
108	Drainage Canal	191	1240 (101)	2170 (102)	3870 (105)	5200 (98)	6070 (101)	6480 (100)
109	Soap works waste	193	1260 (102)	2100 (99)	3770 (103)	5090 (96)	5910 (99)	6660 (103)
110	Gas plant waste	195	1160 (94)	2000 (94)	3630 (99)	4860 (92)	5470 (91)	6260 (97)
111	Corn Products Co	184	1220 (99)	2020 (95)	3460 (94)	4790 (90)	5460 (91)	6070 (94)
112	"Bubbly Creek," Chicago	192	1300 (106)	2130 (100)	3720 (101)	5160 (98)	5880 (98)	6340 (98)
113	Tan and rye liquor (tannery)	185	1300 (106)	2070 (97)	3630 (99)	4960 (94)	5650 (94)	6270 (97)
114	Soak water (tannery)	187	1400 (114)	2150 (101)	3480 (94)	4730 (90)	5350 (89)	6090 (94)
115	Lime soak (tannery)	211	1230 (100)	1870 (88)	3130 (85)	4120 (78)	4900 (82)	5660 (88)
116	Paint manufacturing waste	190	1110 (90)	1740 (82)	3210 (87)	4200 (80)	5210 (87)	5720 (88)
117 (10)*c*	Spent plating bath	148	1050 (85)	1880 (88)	3500 (95)	4580 (87)	5280 (88)	5610 (87)
117 (20)*c*	Spent plating bath	136	910 (74)	1840 (86)	3250 (88)	4630 (88)	5340 (89)	5580 (85)
	Average of impure waters	181	1230 (100)	2040 (96)	3510 (95)	4760 (90)	5500 (92)	6040 (93)

a 10 per cent more mixing water was used for Sample 117 (20), due to quick stiffening.
b Values are average of 30 tests made from three groups. Same values are given in Tables 11 and 12.
c Percentage of Sample 117 used with Lake Michigan water.

TABLE 14.—TESTS OF 1:3 STANDARD SAND MORTARS—SERIES 138.

Compression tests of 2 x 4-in mortar cylinders and tension tests of briquets.
Tests made in accordance with Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials.
Specimens stored in water until test; tested damp.
Each value is the average of 5 tests made on different days.
The values in parentheses are "Strength Ratios," that is percentages of the average strength for fresh waters.

Mixing Water		Flow	Water, per cent by weight	Tensile Strength, lb. per sq. in. (Briquets)						Compressive Strength, lb. per sq. in. (2 by 4-in. Cylinders)					
Sample	Source			3 days	7 days	28 days	3 mos.	1 year	2⅛ years	3 days	7 days	28 days	3 mos.	1 year	2⅛ years
101	Lake Michigan....	112	10.5	215	255	375	410	415	395	1060	1700	2800	3550	3730	3840
102	Distilled water......	114	10.5	220	290	400	380	420	390	1170	1810	2920	3760	4130	4140
103	Distilled aerated.....	112	10.5	235	260	400	400	430	395	1190	1520	2630	3840	4370	3680
	Average....	113	10.5	220 (100)	270 (100)	390 (100)	395 (100)	420 (100)	395 (100)	1140 (100)	1680 (100)	2780 (100)	3720 (100)	4080 (100)	3890 (100)
104	Synthetic sea water..	115	10.5	225 (102)	280 (104)	370 (95)	370 (94)	400 (95)	400 (101)	1320 (116)	1740 (104)	2660 (96)	3470 (93)	3680 (90)	3510 (90)
105	Medicine Lake, S. D.	110	10.7	230 (105)	300 (111)	415 (106)	425 (108)	420 (100)	395 (100)	1300 (114)	1870 (111)	2730 (98)	3680 (99)	3930 (96)	3990 (103)
106	Industrial sewage....	114	10.5	225 (102)	285 (106)	375 (96)	390 (99)	400 (95)	395 (100)	1100 (96)	1550 (92)	2810 (101)	3640 (98)	4310 (106)	3320 (85)
107	Brewery waste......	113	10.5	220 (100)	290 (107)	360 (92)	390 (99)	390 (93)	395 (100)	1180 (104)	1750 (104)	2800 (101)	3950 (106)	4440 (109)	3900 (100)
108	Drainage Canal......	112	10.5	210 (95)	270 (100)	365 (94)	390 (99)	390 (93)	370 (94)	1080 (95)	1650 (98)	2700 (97)	3740 (101)	4290 (105)	3480 (90)
109	Soap works waste....	116	10.5	210 (95)	265 (98)	370 (95)	410 (104)	430 (102)	365 (92)	1070 (94)	1660 (99)	2810 (101)	3530 (95)	3980 (97)	3870 (99)
110	Gas plant waste.....	114	10.5	210 (95)	280 (104)	365 (94)	400 (101)	385 (92)	370 (94)	1040 (91)	1760 (105)	2720 (98)	3680 (99)	4090 (100)	3620 (93)
111	Corn Products Co...	115	10.5	210 (95)	275 (102)	360 (92)	410 (104)	365 (87)	385 (97)	1190 (104)	1820 (108)	2650 (96)	3760 (101)	4060 (99)	3180 (82)
112	"Bubbly Creek," Chicago....	114	10.5	200 (91)	265 (98)	350 (90)	420 (106)	385 (92)	375 (95)	1110 (97)	1640 (98)	2710 (97)	3580 (96)	3980 (97)	3820 (98)
113	Tan and dye liquor (tannery).........	116	10.5	225 (102)	280 (104)	380 (97)	420 (106)	420 (99)	365 (92)	1130 (99)	1800 (107)	2550 (92)	3380 (91)	3540 (86)	3770 (97)
114	Soak water (tannery)	112	10.5	225 (102)	285 (106)	345 (88)	400 (101)	405 (96)	385 (97)	1230 (108)	1860 (111)	2710 (97)	3130 (84)	3570 (87)	3790 (97)
115	Lime soak (tannery).	135	10.5	210 (95)	265 (98)	320 (82)	340 (86)	355 (85)	385 (97)	1140 (100)	1760 (105)	2380 (86)	3210 (86)	3530 (86)	3140 (81)
116	Paint manufacturing waste...........	114	10.5	220 (100)	270 (100)	365 (94)	350 (89)	400 (95)	405 (102)	1190 (104)	1780 (106)	2890 (104)	3550 (95)	3970 (97)	3570 (92)
117 (20)a	Spent plating bath...	106	11.0	180 (82)	270 (100)	370 (95)	440 (111)	420 (100)	365 (92)	1270 (111)	1830 (109)	3030 (109)	3870 (104)	4460 (109)	4380 (113)
117 (10)a	Spent plating bath...	111	10.8	210 (95)	285 (106)	410 (105)	430 (109)	435 (104)	385 (97)	1250 (110)	1880 (112)	2720 (98)	3960 (106)	4420 (108)	4220 (109)
	Average of impure waters..........	114	10.5	215 (98)	260 (104)	365 (94)	400 (101)	396 (94)	380 (96)	1170 (103)	1760 (105)	2720 (98)	3600 (97)	4020 (99)	3710 (95)

a Percentage of Sample 117 used with Lake Michigan water.

In Series 137, 8 samples may be so classified. Tests on Lake Michigan and distilled water were repeated in different portions of the investigations.

Considerable difficulty was experienced in classifying some of the samples; for instance, Sample 37 form Youghiogheny River, Pa., was said to contain sewage and industrial wastes, but the analysis showed it to be remarkably free from impurities, hence it was grouped with the fresh waters.

In each series the *average* strength of concrete made from the fresh waters was used as a basis for studying the values from impure waters (Table 15). The percentages secured in this way are referred to as the "strength-ratios" of the concrete for a given mix, age, curing condition, etc.

TABLE 15.—SUMMARY OF TESTS OF CONCRETE MIXED WITH FRESH WATER.

Mix, 1:4 by volume. See Tables 10 and 13 and Figs. 3 and 6.
For data on other mixes and consistencies using fresh water see Tables 11 and 12.

Curing Condition	Compressive Strength, lb. per sq. in.					
	3 days	7 days	28 days	3 months	1 year	2⅓ years
Series 137—Relative Consistency 1.10						
Moist room	960	1600	3080	4340	5450	5130
Moist room 28 days, remainder in air	....	....	....	4350	3900	3340
Series 138—Relative Consistency 1.00						
Moist room	1230	2130	3680	5280	6000	6460

The two series have one group of tests in which the concrete was quite similar, that is, the 1 : 4 mix, relative consistency 1.10, tested at 28 days; strengths of 3080 and 3350 lb. per sq. in. were obtained for Series 137 and 138, respectively. This is a satisfactory agreement, considering that a smaller size of test piece and a smaller aggregate were used in the first series, and that the concrete was mixed from different lots of materials about 9 mo. apart.

The higher strengths in Series 138 (Table 15) may be attributed principally to the drier consistency used.

The 8 different samples of fresh water in Series 137 gave essentially the same results throughout the period covered by the tests, that is, 3 days to 2 1/3 yr.; the mean variation of the samples from the average was about 5 per cent.

Distilled Water gave concrete strengths essentially the same as other fresh waters; the slight discrepancies which occur in a few instances may be attributed to accidental variations in the tests.

Impure Waters.—Comments on a given type of water are grouped together and refer to both series of tests. In general comparisons are made

on the basis of the *strength-ratios* as compared with the *average* of the fresh waters used in the same mixes, etc., in the same series.

The average concrete strengths and strength-ratios of each of the 6 groups of waters tested at different ages in Series 137 are plotted in Fig. 3. Similar data for each of the samples in Series 138 are shown in:

Fig. 4 Effect of Quantity of Cement,
Fig. 5 Effect of Consistency of Concrete,
Fig. 6 Effect of Age of Concrete.

Bog Waters.—7 samples of bog waters which were thought to be unsuitable for mixing concrete were collected in Minnesota. The chemical analyses of these waters showed only small quantities of foreign matter; the total solids was frequently lower than in Lake Michigan water. In general, these waters gave good results in concrete at all ages (See Table 10 and Fig. 3). The average strengths for individual waters did not often vary more than 10 per cent from the average for the fresh waters. The mean variation from the average concrete strength for the fresh waters at a given age was about 6 per cent.

TABLE 16.—STRENGTH-RATIOS OF CONCRETE MIXED WITH SEA WATER AND SYNTHETIC SEA WATER.

Mix 1:4 by volume, unless otherwise noted.
Relative consistency 1.10 for Series 137 and 1.00 for Series 138, unlese otherwise noted.
For details, see Tables 10 to 13; also Figs. 4 to 6.

Sea Water			Synthetic Sea Water								
Samples 3 and 16 Series 137			Sample 4—Series 137			Sample 104—Series 138					
						Moist Room Curing					
Age at Test	Moist Room	Air*	Age at Test	Moist Room	Air*	Age at Test	Strength Ratio	Mix by Volume	Strength Ratio	Relative Consistency	Strength Ratio
3 days	124	..	3 days	137	..	3 days	129	1:7	97	0.90	87
7 days	110	..	7 days	93	..	7 days	106	1:5	93	1.00	89
28 days	88	..	28 days	85	..	28 days	89	1:4	89	1.10	85
3 months	80	88	3 months	84	86	3 months	79	1:3	88	1.25	87
1 year	86	100	1 year	83	96	1 year	88	1:2	90	1.50	92
2⅓ years	85	96	2⅓ years	88	110	2⅓ years	81				

* Moist room 28 days; remainder in air.

Alkali Waters include all the samples in Group 3 and 4 of Series 137, synthetic sea water and water from Medicine Lake in Series 138. For convenience the discussion of the tests will be presented under the following headings; sea water, synthetic sea water, Great Salt Lake, Devil's Lake, Medicine Lake, waters from tile drains and small streams in sulphate districts and solutions of common salt. Some of the mine and mineral waters (Group 5, Series 137) showed a higher sulphate content than certain samples in Group 3, but it seemed best to preserve this classification.

TABLE 17.—STRENGTH-RATIOS FOR CONCRETE MIXED WITH WATER FROM CHICAGO DRAINAGE CANAL AND ILLINOIS RIVER.

Mix 1:4, by volume.
Series 137 was begun Aug., 1920; Series 138, April, 1921.
For details see Table 10; also Fig. 6.

Age at Test	Strength-Ratio, per cent				
	Illinois River		Chicago Drainage Canal		
	Sample 13 Series 137		Sample 47 Series 137		Sample 108 Series 138
	Moist Room	Air	Moist Room	Air	Moist Room
3 days	101	..	103	..	101
7 days	93	..	96	..	102
28 days	83	..	97	..	105
3 months	85	92	97	105	98
1 year	90	92	94	96	101
2⅓ years	91	102	94	117	100

TABLE 18.—STRENGTH-RATIOS OF CONCRETE MIXED WITH TANNERY WASTES.

For details see Tables 10 to 13, also Fig. 4 to 6.

Mix	Relative Consistency	Water-Cement Ratio	Age at Test	Strength-Ratio, per cent			
				Series 137	Series 138		
				Tannery Waste (42)	Tan and Dye Liquor (113)	Soak Water (114)	Lime Soak (115)
1:7	1.00	1.03	28 days	..	99	97	83
1:5	1.00	0.85	28 days	..	98	94	80
1:4	1.00	0.75	28 days	..	99	94	85
1:3	1.00	0.65	28 days	..	98	92	82
1:2	1.00	0.55	28 days	..	96	94	80
1:4	0.90	0.68	28 days	..	97	93	83
1:4	1.00	0.75	28 days	..	99	94	85
1:4	1.10	0.82	28 days	..	93	92	79
1:4	1.25	0.92	28 days	..	88	93	79
1:4	1.50	1.09	28 days	..	94	86	79
1:4	1.00	....	3 days	99*	106	114	100
1:4	1.00	....	7 days	86*	97	101	88
1:4	1.00	....	28 days	81*	99	94	85
1:4	1.00	....	3 months	86*	94	90	78
1:4	1.00	....	1 year	84*	94	89	82
1:4	1.00	....	2⅓ years	82*	97	94	88

* Relative consistency 1.10; see Table 10.

TABLE 19.—STRENGTH-RATIOS OF CONCRETE MIXED WITH WATER FROM "BUBBLY CREEK".

Mix 1:4; age at test, 28 days, unless otherwise noted.
Moist-room curing, unless otherwise noted.
For details see Tables 10 to 13, also Fig. 4 to 6.

Sample 44—Series 137			Sample 112—Series 138					
Age at Test	Strength-Ratio, per cent		Age at Test	Strength-Ratio, per cent	Mix	Strength-Ratio, per cent	Relative Consistency	Strength-Ratio, per cent
	Moist Room	Air						
3 days......	114	..	3 days	106	1:7	105	0.90	95
7 days......	96	..	7 days	100	1:5	99	1.00	101
28 days......	96	..	28 days	101	1:4	101	1.10	99
3 months.....	100	101	3 months	98	1:3	97	1.25	97
1 year.......	101	91	1 year	98	1:2	97	1.50	100
2⅓ years....	94	100	2⅓ years	98				

TABLE 20.—STRENGTH-RATIOS OF CONCRETE MIXED WITH WASTE FROM CORN PRODUCTS CO.'S PLANT.

Sample 111, Series 138. See Tables 11 to 13 for details; also Fig. 4 to 6.
Mix 1:4; relative consistency 1.00; age at test, 28 days, unless otherwise noted.

Mix	Strength-Ratio, per cent	Relative Consistency	Strength-Ratio, per cent	Age at Test	Strength-Ratio, per cent
1:7..................................	97	0.90	91	3 days	99
1:5..................................	100	1.00	94	7 days	95
1:4..................................	94	1.10	96	28 days	94
1:3..................................	94	1.25	97	3 months	90
1:2..................................	95	1.50	93	1 year	91
				2⅓ years	94

TABLE 21.—STRENGTH-RATIOS OF CONCRETE MIXED WITH WASTE FROM PAINT FACTORY.

Sample 116, Series 138. See Tables 11 to 13 for details; also Fig, 4 to 6.
Mix 1:4; relative consistency 1.00; age at test, 28 days, unless otherwise noted.

Mix	Strength-Ratio, per cent	Relative Consistency	Strength-Ratio, per cent	Age at Test	Strength-Ratio, per cent
1:7..................................	88	0.90	89	3 days	90
1:5..................................	89	1.00	87	7 days	82
1:4..................................	87	1.10	85	28 days	87
1:3..................................	79	1.25	83	3 months	80
1:2..................................	89	1.50	84	1 year	87
				2⅓ years	88

Sea Water.—Series 137 included tests on 2 samples of sea water collected from different points on the New Jersey coast. These samples were handled separately throughout. The average strength-ratios are given in Table 16. For sake of comparison the values for synthetic sea water are included.

Sea water (and synthetic sea water) had an accelerating effect on the early hardening of concrete as shown by the strength-ratios of 124 per cent to 137 per cent at 3 days. At an age of about 10 days, the strength was equal to that of fresh water; from 28 days to 2 1/3 yr., the strength-ratios ranged from 80 to 88 per cent for moist-room curing. The specimens cured in moist room for 28 days and remainder in air showed a decided recovery as indicated by the strength-ratios of 88, 100, and 96 per cent at 3 mo., 1 and 2 1/3 yr. Sea water gave normal results in the time of setting tests.

TABLE 22.—STRENGTH-RATIOS OF CONCRETE MIXED WITH ACID WATERS.

Sample 117, Series 138. See Tables 11 to 13 for details; also Fig. 4 to 6.
Mix 1:4; relative consistency 1.00: age at test, 28 days, unless otherwise noted.
10 and 20 per cent refer to quantity of acid water mixed with water from Lake Michigan.

Mix	Strength-Ratio per cent		Relative Consistency	Strength-Ratio per cent		Age at Test	Strength-Ratio per cent	
	10	20		10	20		10	20
1:7	120	108	0.90	90	93	3 days	85	74
1:5	109	99	1.00	95	88	7 days	88	86
1:4	95	88	1.10	94	94	28 days	95	88
1:3	93	83	1.25	97	97	3 months	87	88
1:2	89	84	1.50	106	104	1 year	88	89
						2⅓ years	87	85

Synthetic Sea Water.—A 3.5 per cent solution of salts consisting principally of sodium chloride and magnesium sulphate in fresh water is frequently used in experimental work as a substitute for natural sea water. (For analysis, see Tables 2 and 4.) The parallel tests for moist-room curing in Series 137 and 138 show a close agreement considering that different lots of materials were used, etc. The specimens cured in air showed about the same increase in strength-ratios as the natural sea water. Tables 11 and 12 show that neither mix (quantity of cement) nor the consistency of the concrete (quantity of mixing water) exerted much influence on the strength-ratios of concrete made from synthetic sea water at 28 days. The strength-ratios are summarized in Table 16. Synthetic sea water produced the same effect in concrete as natural sea water.

Numerous instances are on record in which sea water was used in mixing concrete with apparently good results; however, experienced engineers are practically unanimous in the belief that sea water should not

be used for mixing in *reinforced-concrete work*, particularly in the tropics.*

Great Salt Lake Water (Sample 5) contained about 20 per cent of salts in solution. Analyses of this water reported by different authorities show concentrations ranging from 14 to 27 per cent.† In general the salts

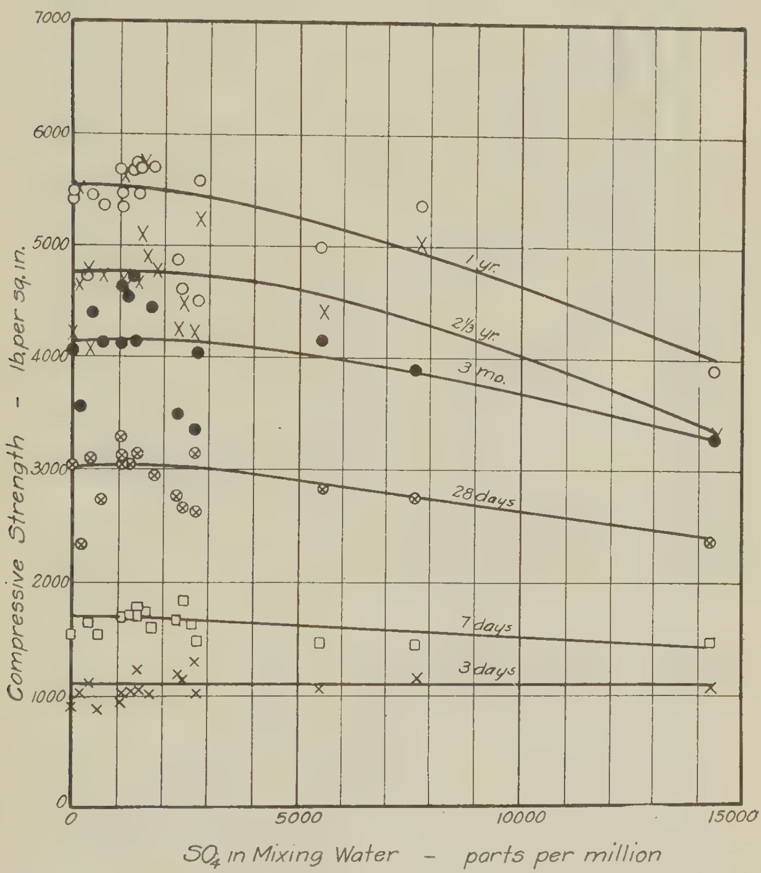


FIG. 1.—EFFECT OF SO_4 IN MIXING WATER ON THE STRENGTH OF CONCRETE.
(Data from Series 137.)

Compression tests of 4 x 8-in. concrete cylinders, cured in moist room. Mix 1 : 4 by volume; relative consistency 1.10; aggregate graded 0-¾-in. Cement: a mixture of 5 brands of portland cement purchased in Chicago. Each value is the average of 5 or 6 tests made on different days.

---

*See "Concrete Viaducts on the Key West Extension of the Florida East Coast Railway," by G. P. Carver; Eng. Record, Oct. 20, 1906.

"Bad Effects Resulting from Use of Salt Water in Reinforced-concrete Structures Built in Tropical Countries," by J. L. Harrison; Abstract Eng. News, v. 76, p. 1047, Nov. 30, 1916.

†"Data on Geochemistry," by F. W. Clarke, Bull. 695, U. S. Geological Survey, 1920.

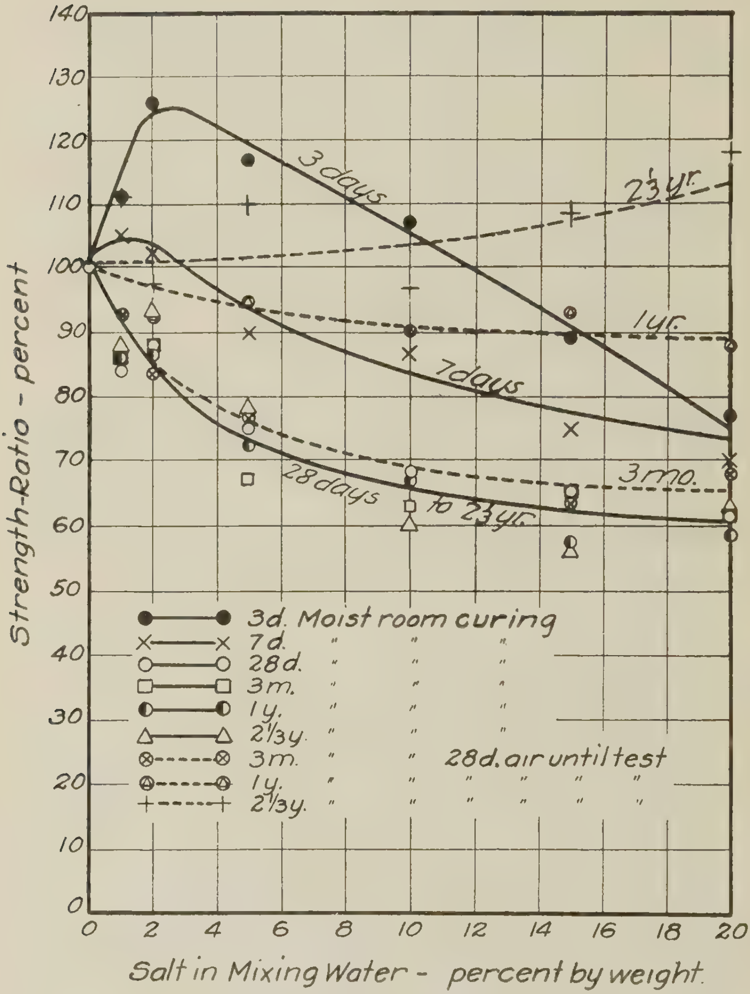


FIG. 2.—EFFECT OF COMMON SALT IN MIXING WATER ON THE STRENTH OF CONCRETE.

(Data from Series 137.)

Compression tests of 4 x 8-in. concrete cylinders. Mix 1 : 4 by volume; relative consistency 1.10; aggregate graded 0-¾-in. Each value is the average of 5 or 6 tests made on different days.

consisted of sodium chloride, although magnesium and calcium salts and sulphates were present. The strength-ratios were 112, 91, 77, 76, 71 and 65 per cent at 3, 7 and 28 days, 3 mo., and 1 and 2 1/3 yr., respectively. This water is not considered satisfactory for mixing concrete, unless we

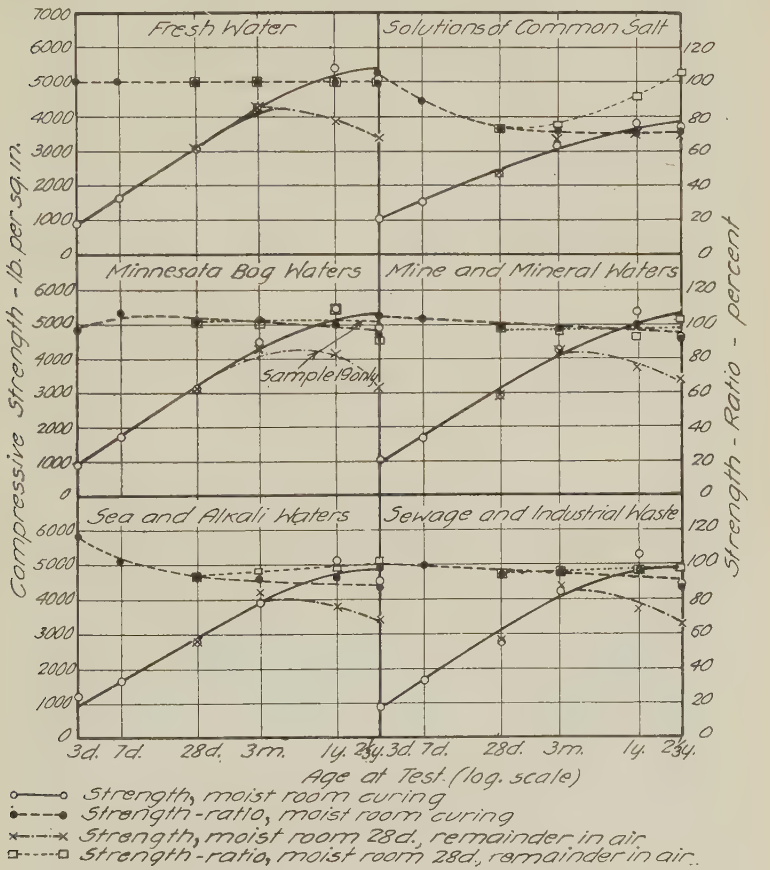


FIG. 3.—EFFECT OF TYPE OF MIXING WATER ON THE STRENGTH OF CONCRETE. (Data from Series 137.)

Compression tests of 4 x 8-in. concrete cylinders. Mix 1 : 4 by volume; relative consistency 1.10; aggregate graded 0-¾-in. Each value is the average for all of the waters in a given group.

take into account the fact that the concrete strength will be reduced to about 70 per cent of normal.

Devil's Lake Water, N. D. (Sample 26) contained about 0.4 per cent of salts in solution, largely sodium sulphate and sodium chloride in about equal amounts. This water gave strength-ratios of 102, 107, 99, 96, 98 and 90 per cent for the ages mentioned above.

Medicine Lake Water, S. D. (Sample 105) consists of about 3.5 per cent solution of sulphates, largely magnesium and gave strengths about the same as sea water. The strength-ratios were 110, 102, 88, 84, 88 and 92 per cent.

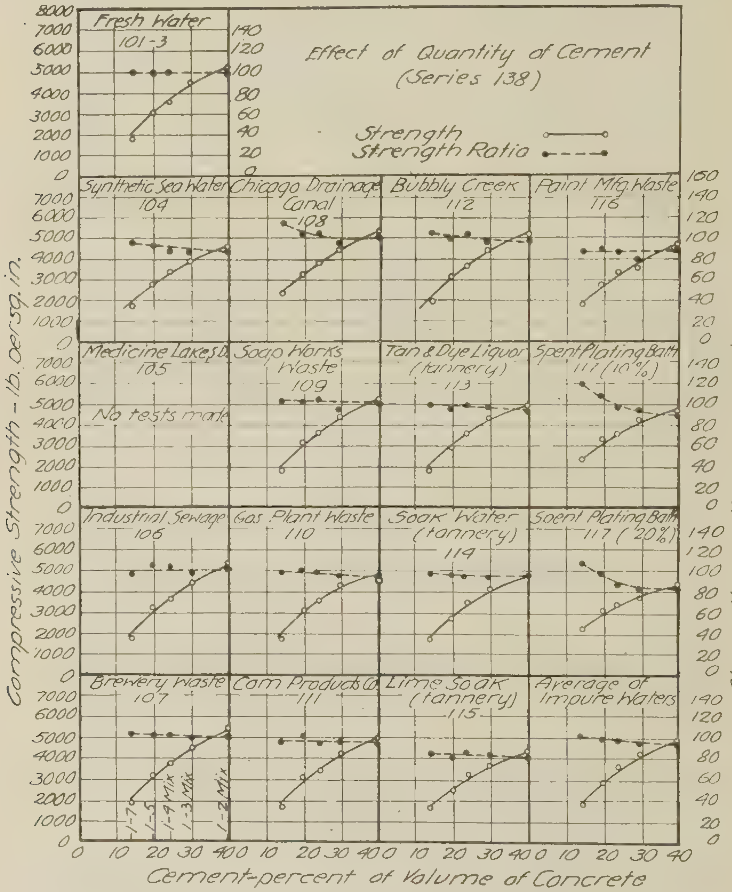


FIG. 4.—EFFECT OF QUANTITY OF CEMENT ON THE STRENGTH OF CONCRETE MIXED WITH IMPURE WATERS.

(Data from Series 138.)

Compression tests of 6 x 12-in. concrete cylinders, cured in moist room. Mix by volume; relative consistency 1.00; aggregate graded 0-1½ in. Cement: a mixture of 5 brands of portland cement purchased in Chicago. Age at test 28 days. In general, each value is the average of 10 tests made on different days.

Waters from Tile Drains and Small Streams in Alkali Districts.—Samples of water from tile drains and small streams were collected in the alkali districts of Minnesota, Colorado, Utah and New Mexico. (5 samples from Group 3, Series 137.) The SO_4 content of these waters ranged from

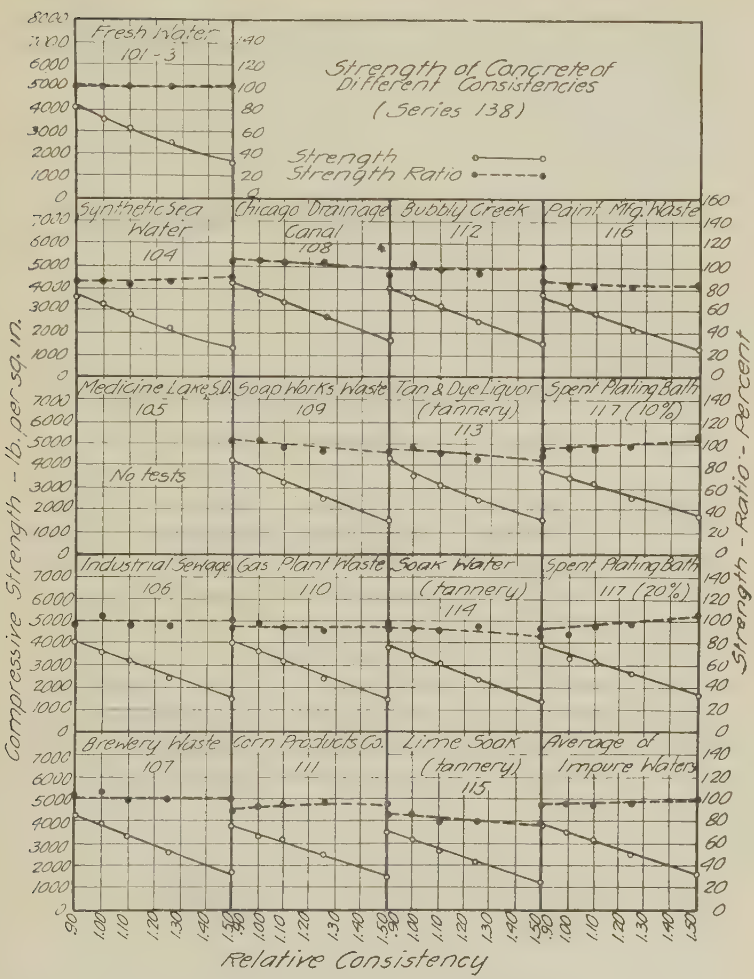


FIG. 5.—EFFECT OF CONSISTENCY ON THE STRENGTH OF CONCRETE MIXED WITH IMPURE WATERS.

(Data from Series 138.)

Compression tests of 6 x 12-in. concrete cylinders, cured in moist room. Mix 1 : 4 by volume; aggregate graded 0-1½ in. Age at test 28 days. In general, each value is the average of 10 tests made on different days.

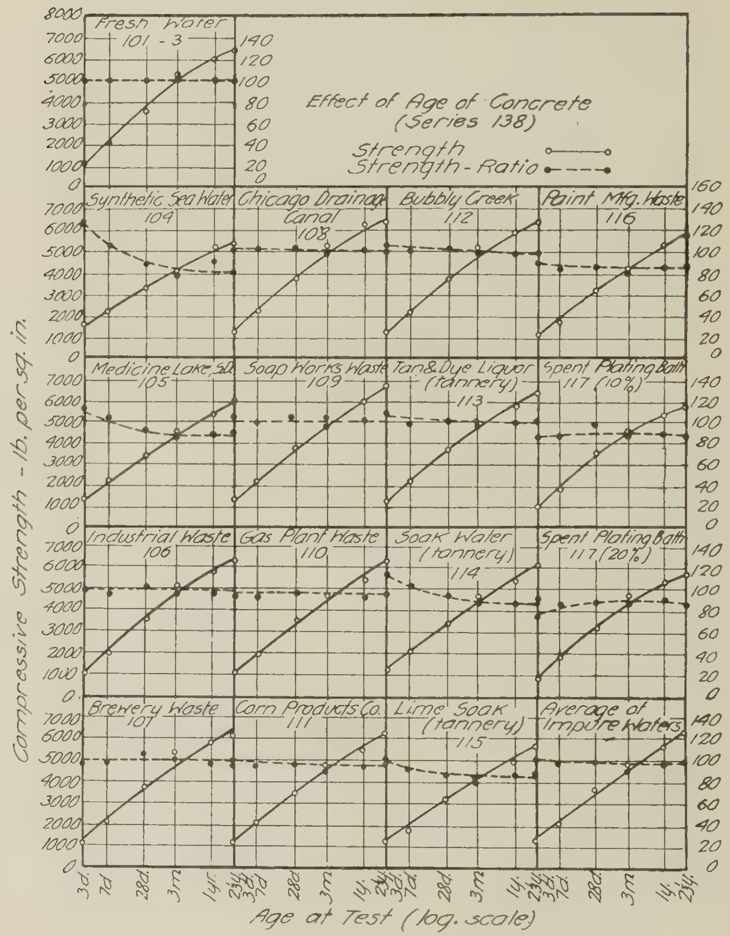


FIG. 6.—EFFECT OF AGE ON THE STRENGTH OF CONCRETE MIXED WITH IMPURE WATERS.

(Data from Series 138.)

Compression tests of 6 x 12-in. concrete cylinders, cured in moist room. Mix 1 : 4 by volume; relative consistency 1.00; aggregate graded 0-1½ in. In general, each value is the average of 10 tests made on different days.

1060 to 7700 parts per million. These samples gave satisfactory concrete strengths at ages up to 2 1/3 yr.; the average strength-ratios were 115, 103, 99, 98, 100 and 94 per cent. The values for air-curing were some-

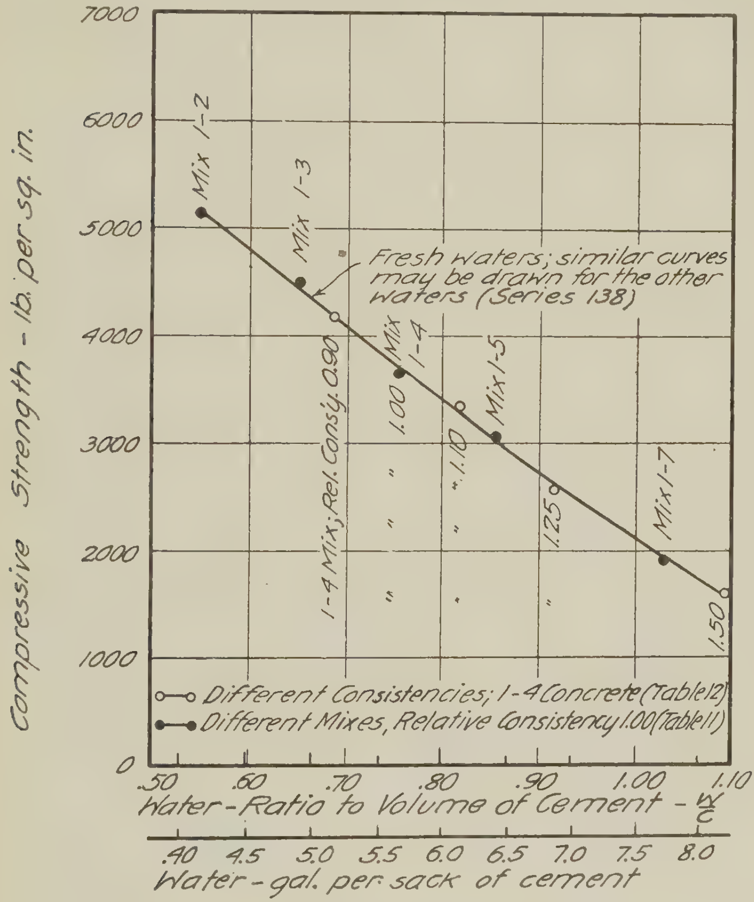


FIG. 7.—EFFECT OF QUANTITY OF MIXING WATER ON THE STRENGTH OF CONCRETE.

(Data from Series 138.)

Compression tests of 6 x 12-in. concrete cylinders, cured in moist room. Mix by volume; aggregate graded 0-1½ in. Age at test 28 days. In general, each value is the average of 30 tests.

what higher than for moist-room curing. The lowest values were found for Samples 31 and 46 from Utah.

Effect of SO_4 Content of Mixing Water.—In Fig. 1 a study has been made of the relation between the strength of 1 : 4 concrete and the SO_4

content of the mixing water for all the samples in Series 137. The diagram shows that, on the whole, little or no ill effects are produced by sulphates until the concentration of SO_4 in mixing water reaches 1.0 per cent. For 0.5 per cent the average reduction in strength as shown by the curves was 4 per cent; a concentration of 1 per cent (10,000 parts per million) was required to produce a reduction in strength of concrete of more than 10 per cent.

Solutions of Common Salt.—Tests were made using solutions of common salt (sodium chloride) in concentration up to 20 per cent as mixing water. Tests were made at ages of 3, 7 and 28 days, 3 mo., 1 and 2 1/3 yr. for specimens cured in moist room and at ages of 3 mo., 1 and 2 1/3 yr. for specimens cured 28 days in moist room and remainder in air. Concentrations of 1 and 2 per cent of common salt produced a slight increase in strength at early ages; at later ages the strength-ratios were below "par," even for these low concentrations. Data of the tests are plotted in Figs. 2 and 3. There was quite a wide difference due to curing conditions. For moist-room curing, even for the low concentrations, the strength-ratios decreased rapidly as the concentration was increased and as the age at test increased. Concentrations of 5, 10, 15 and 20 per cent salt, gave strength-ratios at 28 days of 75, 68, 65 and 61 per cent; the ratios at ages of 3 mo. to 2 1/3 yr. were in general still lower.

The tests on air-cured specimens gave results at variance with those secured for moist-room curing; at ages of 1 and 2 1/3 yr. the strengths for air-curing compared favorably with that of the fresh water.

There is an old rule for concrete work in cold weather to the effect that freezing of concrete can be prevented by adding to the mixing water 1 per cent of salt for each degree Fahrenheit below the freezing point. Such a rule is to be found in many text-books published during the past 40 years.*

The following values† indicate the freezing point of water containing different quantities of common salt:

Salt in Water Per Cent by Weight	Freezing Point of Solution °F.	Strength-Ratios of Concrete 7 days	Strength-Ratios of Concrete 28 days to 2 1/3 yr.
0	32.0	100	100
1	30.5	104	93
2	29.3	103	85
5	25.2	94	72
10	18.7	84	66
20	6.1	75	60

*This rule is attributed to Tetmajer, by Taylor and Thompson; see "Concrete Plain and Reinforced," 1916, p. 287.

†"Mechanical Engineers Handbook," by Marks.

The strength-ratios are from the curves for moist-room curing in Fig. 2.

Five per cent of common salt lowers the freezing point of water about 6° F., but *reduces the strength* of the concrete nearly 30 per cent. Due to the serious reduction of strength by solutions of common salt, their use in mixing water should not be permitted.

Mine and Mineral Waters.—7 samples of water were collected from mines in Arizona, Illinois, Iowa, Pennsylvania, West Virginia and from a mineral spring in Colorado. Chemical analyses of these waters showed them to contain solids ranging from 400 to 4640 parts per million. It is interesting to note that pumpage waters from coal and gypsum mines made good concrete. The only water in this group which gave abnormal values was Sample 12, from a mineral spring in Montrose, Colo. This water was highly carbonated when discharged. Its strength-ratios were 106, 100, 77, 83, 85 and 93 per cent. The other waters gave approximately normal strengths.

A mine water (Sample 30) from Bisbee, Arizona, and Sample 32 from Loop Creek, W. Va., which receives pumpage from 24 coal mines, showed a lower proportion of impurities than some of the fresh waters from city water supplies and for this reason might have been included with the fresh waters. Whether these samples are so included makes little difference since their strength ratios were practically 100 per cent.

Sewage and Industrial Wastes.—The waters in Group 6, Series 137, have been classified under this heading, as well as most of the waters in Series 138. These waters may be divided into those containing sanitary sewage and miscellaneous industrial wastes. The waters containing sanitary sewage are discussed below under: Chicago Drainage Canal, Illinois River Water and Monongahela River Water. Undoubtedly many of the waters listed under Industrial Wastes contained sanitary sewage, but they are primarily carriers of wastes from manufacturing plants.

Chicago Drainage Canal carries Chicago's sewage, diluted with flow from Lake Michigan. This water (Samples 47 and 108) was tested in both Series. The chemical analyses showed both samples to be low in solids. In both series of tests this water gave concrete strengths about the same as the fresh waters.

Illinois River Water (Sample 13) was collected near Marseilles. In spite of the fact that this stream carried Chicago's sewage, the chemical analysis shows comparatively little foreign matter; the total solids were less than for many of the fresh waters. The concrete tests using this water gave results not materially different from the fresh waters; while strength-ratios as low as 83 per cent were obtained at 28 days, it is probable that these variations were more or less accidental, since the ratios increased at 2 1/3 yr. to 91 and 102 per cent for moist room and air curing. Strength-ratios for waters from the Chicago Drainage Canal and Illinois River are summarized in Table 17.

Monongahela River Water (Sample 39) was comparatively free from foreign material and gave strengths approximately normal.

Allegheny River Water (Samples 41 and 48) gave strengths approximately normal. There was no essential difference in the results obtained from the low water and high water samples.

Industrial Wastes.—Waters carrying concentrated wastes from many different industries were tested in both Series. The tests are discussed under:

Refuse from oil refinery,
Tannery wastes,
Waters from "Bubbly Creek", Chicago,
Brewery waste,
Soap works waste,
Gas plant waste,
Waste from Corn Products Refining Co.,
Paint factory waste,
Acid waters,
Miscellaneous wastes.

Refuse from Oil Refinery from Franklin, Pa., (Sample 40) contained large quantities of chlorides, sulphates, calcium and magnesium. The strength-ratios were 66, 115, 94, 88, 83 and 95 per cent. The low value at 3 days is explained by the extremely slow setting of the cement; Table 8 shows that initial setting was delayed to about 10 hr. and final setting to over 36 hr. Normal consistency of the cement with this sample was 26 per cent as compared with 22.5 per cent for fresh waters.

Tannery Wastes.—Three samples of waste from a tannery in Chicago were tested in Series 138; Sample 42 in Series 137 from Endicott, N. Y., probably contained tannery wastes. The principal impurities appear to be calcium chloride and small quantities of sulphates. The strength-ratios for these samples are summarized in Table 18. Tan and dye liquor (Sample 113) gave strength-ratios of from 96 per cent to 99 per cent for concrete mixtures of 1: 7 to 1: 2 at 28 days. For relative consistencies of 0.90 to 1.50 in a 1: 4 mix at 28 days this sample gave strength-ratios ranging from 88 to 99 per cent. In 1: 4 concrete (relative consistency 1.00) at ages of 3 days to 2 1/3 yr., the strength-ratios were in no cases less than 94 per cent.

Soak Water (Sample 114) gave strength-ratios for the different conditions of test ranging from 86 per cent to 114 per cent, average 94 per cent.

Lime Soak (Sample 115) gave consistently lower strength-ratios than Samples 113 and 114. For this water the strength-ratios ranged from 78 to 100 per cent and were generally less than 85 per cent. The strength-ratios for *Tannery Waste* (Sample 42) were essentially the same as for Sample 115.

"Bubbly Creek" water (Samples 44 and 112) contains waste from the Chicago Stockyards and "Packingtown" and probably represents the most polluted body of water in the world. The odor is extremely offensive. During the summer a scum consisting of grease and other materials forms

on the surface. Concrete tests were made using this water in both series. The strength-ratios are summarized in Table 19 and are plotted in Fig. 4 to 6. For all mixes, consistencies and ages the concrete strengths were essentially the same as that produced by fresh water. Judged by any of the usual standards, this water, prior to these tests, would not have been considered acceptable for mixing concrete, yet the tests show it to give good results.

Brewery Waste (Sample 107) gave concrete strengths almost identical with fresh water for different mixes and consistencies at 28 days and for the 1: 4 mix at different ages.

Soap Works Waste (Sample 109) gave concrete strengths almost identical with that of fresh water.

Gas Plant Waste.—Water used to seal gas machines (Sample 110) gave strength-ratios of about 95 per cent.

Waste from Corn Products Refining Co. (Sample 111) gave strengths which compared favorably with fresh waters. In no case were the strength-ratios below 90 per cent and averaged 94 per cent (Table 20).

Paint Factory Waste (Sample 116, see Table 21 and Fig. 4 to 6) gave strength ratios ranging from 79 per cent to 90 per cent, average about 86 per cent. The low strength from this water is probably due to the presence of acid.

Acid Water from a spent plating bath (Sample 117) contained sulphuric acid in such a high concentration that it was necessary to dilute it to 10 and 20 per cent of its original concentration by the addition of Lake Michigan fresh water before it could be used for mixing concrete. The effect of this water varied considerably with the concentration and the characteristics of the concrete with reference to quantity of cement, mixing water, etc. The 1: 7 mix at 28 days showed strength-ratios of 108 and 120 per cent while the 1: 2 mixtures gave strength-ratios of about 86 per cent. For the usual consistencies and mixtures the strength-ratio was about 90 per cent. The water required for normal consistency of cement using the acid waters was 26 and 27 per cent as compared with 24 per cent for fresh water. The time of setting of the cement with these waters was in general somewhat longer than for fresh water.

Effect of Quantity of Cement (Mix).—Fig. 4 shows the relation of the 28-day strength and strength-ratios to quantity of cement for 1: 7 to 1: 2 concrete mixes of relative consistency 1.00 for the waters in Series 138. In all cases there was an increase in strength with increase in amount of cement. The strength-ratio curves are in general horizontal or nearly so, showing that the effect of the various impure waters was similar for the different mixes. The acid waters from a spent plating bath (Sample 117) gave greater reductions in strength in the rich mixes than in the lean.

Effect of Quantity of Mixing Water (Consistency).—The effect of consistency on the 28-day strength of 1: 4 concrete mixed with 15 different samples of water in Series 138 is shown in Fig. 5. The strength curves

show the marked effect of quantity of mixing water on the compressive strength of concrete mixed with both fresh and impure waters.

The individual strength-ratio curves are in general horizontal showing that the effect of the impure waters was the same, regardless of the consistency of the concrete. The strength-ratio curves for acid waters from a spent plating bath (Sample 117) have an upward trend, showing that the effect of these waters was somewhat less in the wetter concretes. The *average* strength-ratio curve for all impure waters was approximately horizontal.

Fig. 7 shows the relation of quantity of mixing water, as expressed by the water-cement ratio and gallons per sack of cement to the 28-day compressive strength for the fresh waters in Series 138. In these tests the quantity of mixing water was varied by

(a) Change in mix (relative consistency 1.00 (Table 11).

(b) Change in consistency (1: 4 mix, Table 12).

All of the plotted points for the two conditions of test lie close to the curve, showing remarkably consistent results. Similar curves may be drawn for the impure waters in Series 138.

These tests show the importance of the water-ratio-strength relation which has been pointed out in many other reports from this Laboratory. The relation between strength and quantity of mixing water for *fresh waters* is shown below:

Relative consistency of concrete	0.90	1.00	1.10	1.25	1.50
Water-ratio	0.68	0.75	0.82	0.92	1.09
Gallons of water per sack of cement	5.1	5.6	6.2	6.9	8.2
Concrete Strength at 28 days lb. per sq. in.	4200	3680	3350	2580	1190
Strength-ratio to Consistency 1.00	114	100	91	70	43
Strength-ratio to Consistency 1.10	126	110	100	77	47

In computing water-ratios, absorption of aggregates was disregarded. 6.2 gal. of water per sack of cement (water-ratio 0.82) is about the best consistency for concrete road construction; adding 0.7 gal. per sack of cement had the effect of reducing the strength of concrete to such an extent as to give a strength-ratio of 77 per cent.

Increasing the quantity of mixing water produced exactly the same effect as *reducing* the quantity of cement in the batch. (See Fig. 7.) A 1: 4 mix with relative consistency 1.25 (25 per cent more water than "normal" consistency) gave about the same strength at 28 days as a 1: 6 mix of normal consistency; a 1: 4 mix with relative consistency 1.50 gave a lower strength than a 1: 7 mix of normal consistency. A relative consistency of 1.50 is by no means as "sloppy" as concrete frequently seen.

A comparatively slight increase in *quantity* of mixing water may produce a greater reduction in concrete strength than that caused by using the *most polluted* mixing water that is ordinarily encountered.

Effect of Age.—The effect of age on the strength of concrete is shown in Tables 10, 13 and 15, and in Fig. 3 and 6. The results of tests in Series 137 are given in Fig. 3 in which age is plotted to a logarithmic scale. Separate curves are shown for the average of each group of waters for two methods of curing.

Fig. 6 shows the effect of age for the different waters in Series 138. The curves show that unusually consistent results were obtained with each water. In general there was an increase in strength with age for each of the waters used. The strength-ratio-age curves show the relation of strength of concrete made with impure waters as compared with that of similar concrete made with fresh water at the different ages.

Strength-ratios for the fresh waters in the two series, based on the 28-day tests of concrete for specimens cured in the moist room were as follows:

	3 days	7 days	28 days	3 mo.	1 yr.	2 1/3 yr.
Series 137	31	52	100	141	177	167
Series 138	33	57	100	144	163	176

It will be noted that the 1-yr. tests in Series 137 appear to be abnormally high; this may account for the apparent retrogression in the 2 1/3 yr. tests in this Series.

For concrete cured in a damp condition, the strength is approximately proportional to the logarithm of the age at test.

Specimens in Series 137 cured in moist room for 28 days and then in air gave approximately the same strengths at 3 mo. as similar concrete cured in moist room for 3 mo.; at 1 and 2 1/3 yr. air-cured specimens showed a falling off in strength, except in the case of 5 to 20 per cent salt solutions, where increases in strength were generally found.

In Series 138, synthetic sea water (Sample 104), paint manufacturing waste (116), acid waters from a spent plating bath (117) and lime soak (115) gave strength-ratios, in certain instances below the 85 per cent which we have set as the lower limit for acceptable mixing waters. It is interesting to note that the *average* concrete strength for all impure waters in this series was essentially the same as for fresh waters.

The 28-day strength of concrete or mortar seems to be the best indication of quality of mixing water. In a few instances the 3 and 7-day strengths were abnormally high or low. In no instance in the moist room-cured specimens, does the later tests give results which are materially at variance with those at 28 days.

Mortar Tests.—Results of tension and compression tests of 1: 3 standard sand mortars at ages of 3 days to 2 1/3 yr. for the waters in Series 138 are given in Table 14. A comparison of these strength-ratios with those for the concrete tests in Table 13 shows that in general the effect of the impure waters on *mortar strength* was similar to that on the concrete strength at different ages. The mortar strength-ratios of synthetic

sea water, Medicine Lake, paint manufacturing waste and acid waters from a spent plating bath were generally somewhat higher at all ages than the strength-ratios of the concrete tests. The strength-ratios for briquet tests of 1-3 standard sand mortar mixed with water from "Bubbly Creek" were somewhat lower than the strength ratios for the compression tests of mortar and concrete.

Normal Consistency.—The percentage of water required for normal consistency of cement when mixed with the different impure waters was in general about the same as for fresh waters. In Series 137 the average normal consistency for the fresh water was 22.5 per cent. Great Salt Lake water (Sample 5) required 24 per cent; solutions of 5 to 20 per cent of common salt required from 24 to 25 per cent. Refuse from an oil refinery (Sample 40) required 25 per cent of water for normal consistency.

In Series 138 the fresh-water normal consistency was 24.5 per cent; most of the impure waters in this series required a similar amount. Medicine Lake water (Sample 105) and acid waters from a spent plating bath of 10 and 20 per cent concentration required 25, 26, and 27 per cent, respectively.

Time of Setting.—In general, the time of setting of cements mixed with impure waters in a given series was about the same as for fresh water. None of the samples gave time of setting decidedly shorter than fresh waters. Sea water and synthetic sea water were somewhat more quick-setting in the Vicat test; the other tests were practically normal.

The following table shows the samples which gave abnormally long time of setting as compared with fresh waters:

Mixing Water	Sample No.	Time of Setting			
		Vicat Needle		Gillmore Needle	
		Initial	Final	Initial	Final
		h. m.	h. m.	h. m.	h. m.
SERIES 137					
Fresh		3 52	7 20	4 43	7 18
Great Salt Lake	5	6 45	11 15	7 30	11 15
10 per cent salt solution	9	4 25	9 30	5 45	9 45
15 per cent salt solution	10	5 00	9 30	6 30	9 45
20 per cent salt solution	11	6 00	10 30	9 30	.. ..
Refuse from oil refinery	40	11 00	36 00	9 00	48 00
SERIES 138					
Fresh	†	3 55	8 30	6 25	9 45
Industrial sewage	106	5 00	9 15	7 30	11 50
Corn products waste	111	5 30	9 20	7 50	11 15
Spent plating bath (10 per cent)	117	4 50	7 55	7 15	12 20
Spent plating bath (20 per cent)	117	5 45	11 15	8 05	13 00

* Average of 8 samples.
† Average of 3 samples.

Refuse from oil refinery (Sample 40) was very slow in setting (average 10 hr. initial and 42 hr. final). While the concrete tests at 3 and 7 days were somewhat erratic, the strength-ratios at ages of 28 days to 2 1/3 yr. ranged from 83 to 95 per cent, average 90 per cent, indicating that this water is satisfactory for mixing concrete.

The time of setting with Industrial Sewage water (106), Corn Products Waste (111) and acid waters from a spent plating bath (117) was somewhat longer than for the fresh waters, as shown by the above table.

The tests show that while the low-strength waters generally gave longer time of setting, there was no direct relation between time of setting and the strength-ratios of the different water samples. Time of setting is not a satisfactory criterion of suitability of a water for mixing concrete.

Soundness.—Soundness tests of the cement mixed with various waters were made in both series; in every case the cements passed the test satisfactorily.

Color and Odor of Waters.—Most of the waters in Series 137 were clear and colorless. The Minnesota bog waters had a yellow color due to decaying vegetation; water from a creek at Lakehurst Air Station, N. J. (Sample 38) and from the Monongahela River (39) were also yellow in color. Refuse from an oil refinery (40) was black and oily. "Bubbly Creek" water (44) was very dark in color.

The impure waters in Series 138 varied in color. Brewery waste (Sample 107), Drainage Canal water (108) Soap Works waste (109) and Corn Products waste (111) were light yellow in color. Gas plant waste (110) had a brown color and a turbid appearance, probably due to globules of oil in suspension. Tannery wastes were turbid; Soak water (114) and Lime soak (115) were white in color, probably due to lime in suspension. Lime soak contained some hair from hides. Paint manufacturing waste (116) was a deep red color and quite clear. Water from a spent plating bath (117) was dark brown and somewhat turbid. "Bubbly Creek" water (44 and 112) was very dark in color; a greasy scum collected on this water after standing for some time.

Few of the waters used had a pronounced odor. "Bubbly Creek" water gave off a very offensive odor; waste washings from beer storage vats (Sample 107) had a beer-like odor; a fresh water from the city water supply, Dundee, Mich., (18) and water from a creek at Lakehurst, N. J., (38) had an odor of hydrogen sulphide. Tannery wastes had a characteristic odor.

The tests show that neither color nor odor furnish a satisfactory criterion of the suitability of a water for mixing concrete.

Specifications for Quality of Mixing Water.—The following extracts from State Highway Specifications are typical of the requirements for quality of water for mixing concrete in road work:

> (a) "All water used in concrete shall be subject to the approval of the Engineer and shall be reasonably clear, free from oil, acid, or vegetable substances and neither brackish nor salty." (Winconsin and Utah.)

(b) "Water used for this work shall be reasonably clean, and free from oil, acid, alkali or vegetable substances." (Indiana.)

(c) "Water used for this work shall be reasonably clean, and free from oil, acid, alkali or vegetable substances. No salt or sea water shall be used in mixing any concrete." (California.)

(d) "The water shall be clean, free from oil, acid, alkali or vegetable substances, and the tensile strength of 1-3 mortars shall be equal to that developed with distilled water when mixed in the same proportion with the same cement and Ottawa sand." (Pennsylvania.)

(e) "Water used in concrete shall be free from sewage, oil, acid, strong alkalis or vegetable matter and also shall be free from clay and loam." (Ohio.)

(f) "Water used in concrete or in grout filler, shall be clean and free from oil, acid, alkali or vegetable matter. Before installing his water supply equipment, the contractor shall secure the Engineer's approval of the source of water supply which he proposes to use. If at any time the water from this source should become of unsatisfactory quality or insufficient quantity the Engineer may require the contractor to provide water from some other source." (Iowa and Minnesota.)

(g) "The water used in mixing portland cement concrete shall be clean, free from oil and organic matter, and when tested with litmus shall show no acid or alkaline reaction." (Illinois.)

(h) "Water shall be clean, free from oil, acid, alkali or vegetable matter." (Arizona and Colorado.)

(i) "Water shall be clean, free from oil, acid, alkali, vegetable matter, organic matter and other deleterious substances." (New York.)

(j) "All waters for use in concrete shall be clean, free from oil, acid, alkali, vegetable matter, sea salts, or ingredients that are injurious to concrete." (New Jersey.)

All the above specifications require that waters shall be *free from* impurities of various kinds. If these specifications were rigidly enforced, but few waters would be available for mixing concrete, as small quantities of impurities are found in practically all waters except rain and distilled waters.

The report of the Joint Committee on Standard Specifications for Concrete and Reinforced-concrete gives the following specifications for mixing water:

> "Water for concrete shall be clean and free from injurious amounts of oil, acid, alkali, organic matter or other deleterious substance."

This specification puts the emphasis on *injurious amounts* of impurities, but leaves open the methods of test.

Tests for Mixing Waters.—In only rare instances are definite tests specified for mixing waters. The U. S. Bureau of Public Roads recommends the following tests:

(1) Acidity and alkalinity, by use of blue and red litmus paper,

(2) Total solids and inorganic matter,

(3) Soundness, time of setting or mortar strength.

"Any indication of unsoundness, marked change in time of setting or a variation of more than 10 per cent in strength from results obtained with mixtures containing the water of satisfactory quality, shall be sufficient cause for rejection of the water under test."*

The litmus test is not of much value since it responds to minute quantities of acid or alkali which our tests have shown to be harmless in concrete. The test for total solids is likely to be misleading. Our studies did not reveal a single sample of impure mixing waters which caused unsoundness in the cement.

Concrete or mortar tests are the most satisfactory guides in important cases.

While this investigation did not cover all possible types of water, it seems safe to conclude that any water which is suitable for drinking or for ordinary household use can be accepted without question for mixing concrete. It has been found that many waters which are badly polluted with sewage and trade wastes are not injurious in concrete.

Effect of Sugar in Concrete.—One type of impurity which was not included in this series of tests, but which should be mentioned here, is sugar and related compounds such as alcohol and fruit juices. In general such compounds are fatal to concrete, since they practically destroy the hydraulic properties of the cement. Waters containing even small quantities of such materials should not be used either in mixing or curing concrete without thorough investigation.

RELATED TESTS.

This report includes only one phase of the general problem of the influence of impurities in concrete and does not touch upon the behavior of concrete when *exposed to* various types of waters and soils. Results have already been reported on previous tests on the effect of organic impurities

---

*Standard and Tentative Methods of Sampling and Testing Highway Materials, Bulletin 949, U S. Dept. of Agriculture, 1921.

in sands[1]. Exhaustive tests have been made in this Laboratory on the effect of powdered admixtures in concrete[2]. Field and laboratory tests are now under way on concrete exposed to sulphate soils and waters; including the effect of integral compounds. A summary of tests on the effect of integral water- and alkali-proofing compounds on the strength of concrete has been published[3]. A report has been prepared on the effect of calcium chloride and other soluble salts in concrete[4]. Tests are under way on the use of calcium chloride and similar materials as a curing agency for concrete.

---

[1] Colorimetric Test for Organic Impurities in Sand, Circular 1, Structural Materials Research Lab., 1917. See also, Abrams-Harder Field Test for Organic Impurities in Sands; Report of Committee C-9, Proc. Am. Soc. Testing Materials, 1919.

[2] Effect of Hydrated Lime and other Powdered Admixtures in Concrete; Proc. Am. Soc. Testing Materials, 1920, Reprinted as Bulletin 8, Structural Materials Research Lab., Lewis Institute, Chicago.

[3] Effect of Integral Waterproofing Compounds on the Strength of Concrete; Industrial and Eng. Chemistry, May, 1923.

[4] Forthcoming Proc. Am. Soc. Testing Materials, 1924.

BUILDING FIREPROOF HOMES.

By Paul Hueber.*

Fireproof homes are by no means a new product. Well-to-do men have been building such homes for years, not because they wanted a cheaper house, but because they wanted the best. These were built according to architects' designs and special forms were required for each house. This increased the cost of construction to such an extent that only the well-to-do

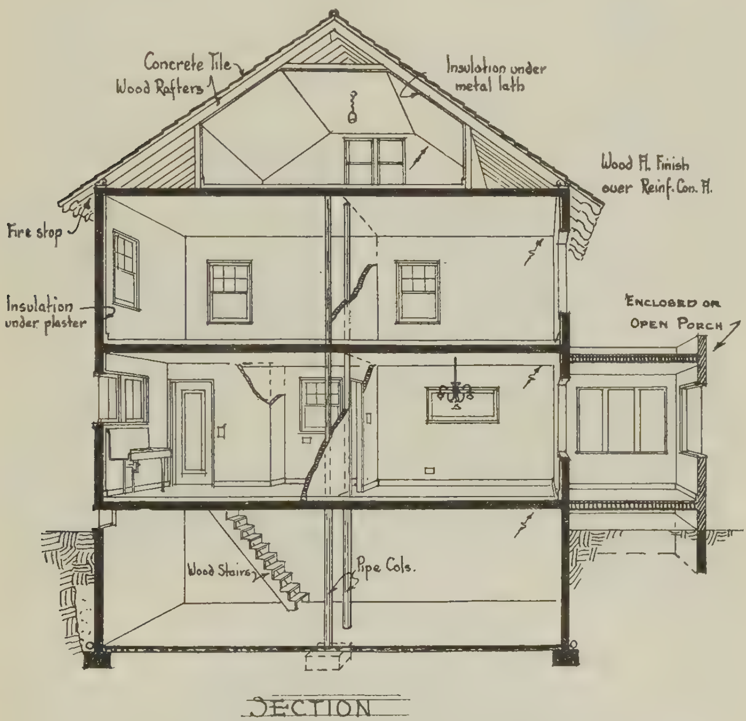


FIG. 1.—EXPOSED SECTION THROUGH TYPICAL HOUSE.

man could afford to own one. The desire to produce lower priced, fireproof homes, (the so-called concrete houses) resulted in the development of standardized forms which in turn resulted in the construction of houses of similar design, which does not necessarily follow.

The section (Fig. 1) gives a view of the problem as we have solved it. We studied the problem first to produce, economically, a permanent, rein-

*Architect, Hueber Brothers, Builders, Syracuse, N. Y.

forced-concrete frame suitable for finishing into any priced home desired. Reducing the frame to a minimum amount of concrete exterior walls and floors meant economy of form work. Interior partitions, just like any modern, fireproof building, are of gypsum tile or cinder block except where plumbing pipes occur. At these points we use wood studs and metal lath. Interior floor supports consist of iron columns. By spacing these close together the reinforced-concrete beams are kept flush with the ceilings. This not only saves beam forms but permits different arrangements of plan with the same floor forms.

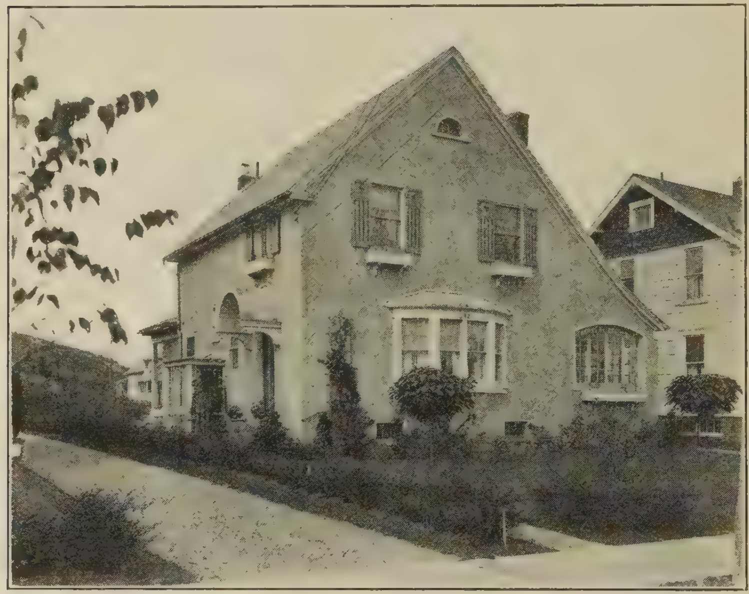


FIG. 2.—ONE OF THE FINISHED HOUSES.

Bay window and porch constructed after completion of frame.

As pipe columns filled with concrete are smaller in section than concrete columns, thinner partitions can be used. Porches are built independent of the frame so as not to complicate the forms. Roofs are built of wood and covered with concrete tile. No attempt is made to secure a hollow wall but insulation is obtained by the use of cork board. Wood floor finishes are used in the majority of our homes so far because the average home buyer is accustomed to them.

Fig. 2 illustrates one of our finished homes. The bay window and porch were constructed after the completion of the concrete frame of the main part of the house. More of our houses built from the same set of forms but with varying roofs, windows, porches and other exterior finish

FIG. 3.—A GROUP OF HUEBER CONCRETE HOUSES.

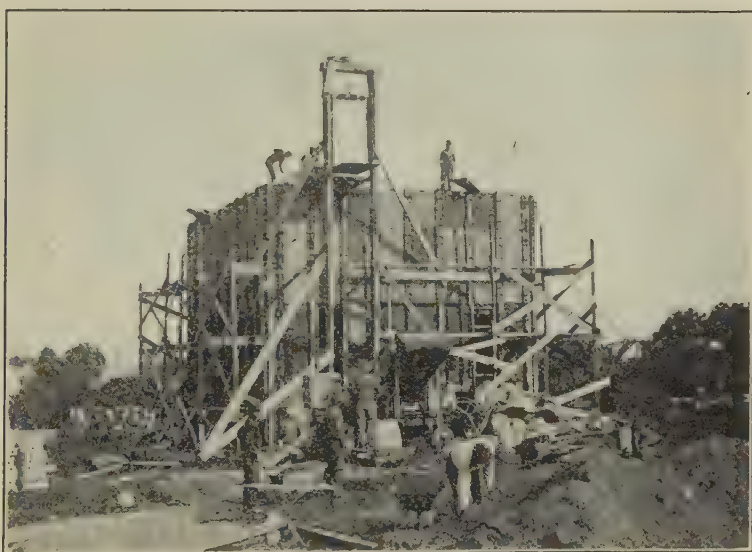


FIG. 4.—POURING ONE OF THE CONCRETE HOUSES.

are shown in Fig. 3. These pictures show that architectural design is not a question of concrete, but is simply a matter of adding certain features to the frame as in the case of a wood house, or for that matter, any building.

FIG. 5.—CONCRETE FACING BEFORE FINISH IS APPLIED.

Fig. 4 shows the lower portion of the walls completed and striped, the exterior forms having been raised and the concrete already placed for the second story walls. The window frame nailed into position can be placed wherever desired, which is one advantage of having wood panel forms. Form panels are in excellent condition after being used 45 times over a 3-year period.

The proposition of depositing concrete in the construction of a house is a different problem than in most other kinds of concrete work owing to the relatively small amount of concrete to be handled. We discharge from the mixer directly into buggies which are raised to the floor above by means of an elevator; elevator runs being attached to forms. No hopper is used in our work. However, we believe that on large operations a central mixing plant from which concrete could be delivered to hoppers at the base of elevators would be a good investment. In our case a 4-cylinder engine, mixer, and hoist are mounted on a truck having rubber tired

FIG. 6.—FIRST CONCRETE HOUSE BUILT BY AUTHOR.

wheels. Chuting of concrete is, of course, not economical on small jobs and with cinder concrete such as we use in our walls above grade, chuting is next to impossible. We pour a story at a time, the same as we do on our other work.

When we started our first house people thought that the concrete frame No. 5 was a concrete house and threatened to stop our work by an injunction. Fig. 6 shows our first concrete house.

Fig. 7 shows what value is built into these homes. The gypsum walls and gypsum tile in floor insure quietness and insulation against heat and cold. The cork lined walls mean a warm house in winter, and a cool house in summer. Our door and window frames are of wood construction as are also the sash and doors.

A wood roof which permits variety in architectural treatment, and, when covered with fireproof shingles (slate, concrete tile) and over a reinforced-concrete attic floor, is safe from fire damage. The porch foundation is of concrete block. Usually, our porch floors are of wood, al-

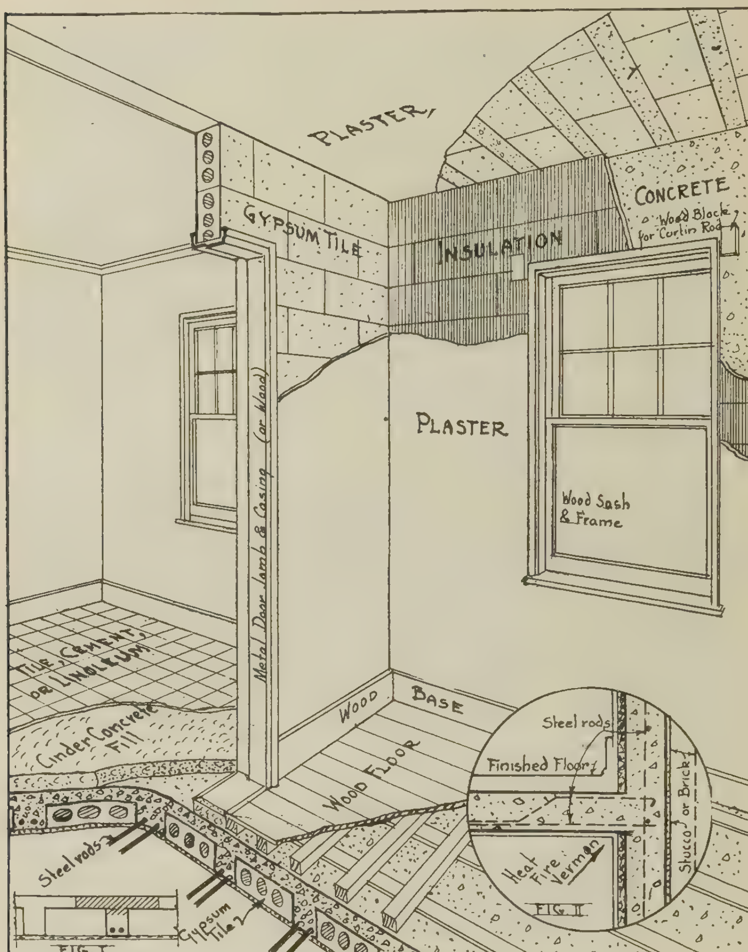


FIG. 7.—EXPOSED SECTION SHOWING CONSTRUCTION.

though in some of our houses reinforced-concrete floors have been built on request. The gable ends are of concrete block construction, being somewhat less costly than if built of monolithic concrete since special forms would be required for each job to take care of the variation in the gables of the different houses.

Stucco adheres tenaciously to the rough surface of cinder concrete. The bond is strong and lasting. In brick veneer construction the brick are laid up after the concrete frame is completed. A thicker wall below grade forms a ledge on the cellar wall to provide a foundation for the brick veneer. Stone concrete is used for walls below grade and for all floors.

In developing our forms, we have not changed the fundamental elements of typical forms. That is, we use sills, studs, plates and joints, but instead of using nails we employ wedges and turn buttons for easy

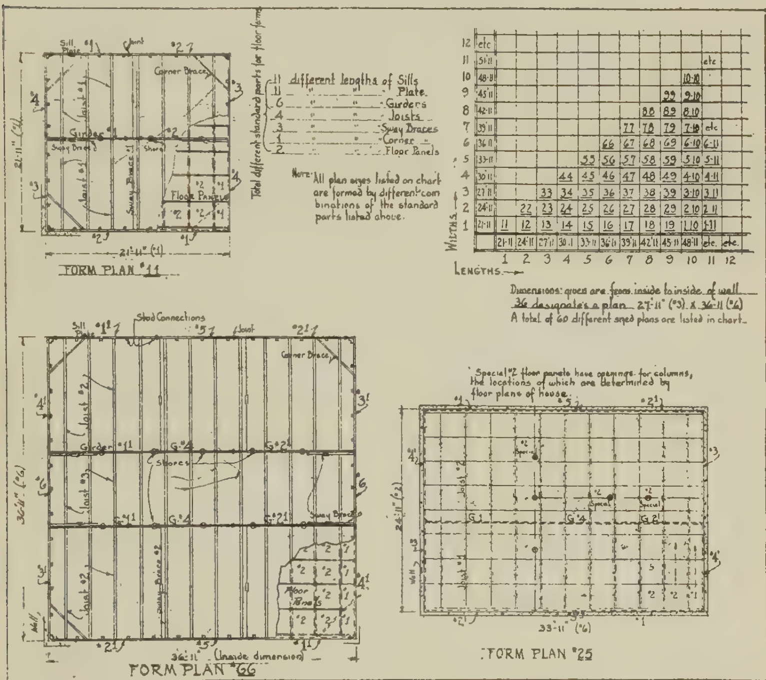


FIG. 8.—DETAILS OF FORMS.

erecting and easy stripping for re-use. Considerable study and actual experience extending over a period of six years resulted in our adoption of these details. The sills, plates and girders are steel channels, the joists are of pressed steel and the studs and panels are of wood. The whole is braced with stiff corner braces and with lateral braces having turnbuckles.

Much effort has been spent by others to obtain air spaces, but in our opinion such a wall is not as effective as a solid wall insulated with cork or Celotex. Reasons for insulating the wall are two: 1. Conserves heat. 2. Prevents condensation.

The floor forms are made adaptable to different sized homes by simply combining a few standard parts. All the standard floor members required are shown in the two form plans. All other sized plans in 3-ft. units are merely different combinations of these parts. The advantage of using closely spaced columns to eliminate beam forms is obvious since form framing is done independent of structural concrete framing. The form framing for each plan is standardized, yet the house can be arranged as desired.

There is a saying, "Watch what enters the construction of homes of well-to-do and public buildings and you will see the future construction of the average man." Public buildings contain practically no wood in their construction. Our homes are very close to public buildings and homes of the well-to-do in this respect.

The two things which have held back the monolithic home are said to have been architecture and cost. We feel that we have demonstrated that architectural designs are readily obtained and our costs not at all alarming, the extra cost being more in the finishing of the concrete frame than in the frame itself, provided the fireproof idea is carried throughout. The cost is naturally more than for a wood frame home, but a fireproof home is worth more and people recognize this value, the same as they do of a fireproof office building.

As the problems of a poured concrete house become better understood, we feel sure that more builders will construct such homes, especially in view of the rapidly diminishing timber supply and abundance of cement, sand, and stone.

Then our homes in the United States will be as permanent and safe from fire as our modern public buildings. This means low upkeep cost, fire protection, insulation against noises in rooms and the elimination of rats, and mice minimum, cracking of plaster, a house warm in winter, and cool in summer.

A STUDY OF BENDING MOMENTS IN COLUMNS.

By F. E. Richart.*

INTRODUCTORY.

Information on the behavior of reinforced-concrete buildings necessary to the proper design of such structures has been accumulated largely in the last twelve or fifteen years. The reinforced-concrete building is a highly indeterminate structure, subject at best only to approximate analysis, but due to the numerous laboratory tests and to investigations of full-sized structures that have been made, there is probably more information now available on the action of the reinforced-concrete structure as a whole and of its elementary parts than there is on other types of construction. In field tests, attention has usually been directed to the behavior of beams, girders, or flat slabs, leaving the columns to a place of secondary consideration. The conditions of laboratory investigations, on the other hand, usually lend themselves to a study of the action of various types of reinforcement on beams, slabs, or axially loaded columns. Data on the amount and distribution of the bending stresses induced in building columns, and on the resistance of columns to the combined bending and direct stress, are rather meager. Due to this lack of information or to other causes, bending stresses are usually neglected in the design of concrete columns, and such stresses are assumed to be provided for, together with numerous other effects, in the all-inclusive factor of safety. That buildings are being built under these conditions, and that they stand up under loads without serious damage, does not prove that better methods of design should not be used when available. No very extended examination of reinforced-concrete buildings is necessary to demonstrate that the average wall column contains a number of cracks which from their position are obviously produced by bending stresses. Such cracks are often unsightly; on the exterior face of the column they may affect its durability. They may be compared both as to cause and to remedy to the cracks often seen in beams and slabs in which reinforcement for negative moment has been omitted.

It is the object of this paper to show the amount and distribution of moments in building columns under certain conditions as indicated by analysis and by the results of experiments, as well as to make a study of the resistance of columns subjected to both bending and axial stresses.

There are at least four general ways in which bending moments are induced in the columns of a building, (a) by reactions from eccentrically placed beams, or from brackets or cantilevers, (b) by eccentricity of the

*University of Illinois, Urbana, Illinois.

column itself such as occurs at changes in section of wall columns in which the exterior face is kept vertical throughout its height, (c) by the action of wind pressure which must be resisted through the stiffness of connections between the columns and the girders or slabs, and (d) by transfer from slab or girder of unbalanced moment produced by floor loads. Of these four cases it may be noted that the first two are generally statically determinate, since the amount of load and the eccentricity are known. The calculation of wind stresses in reinforced-concrete buildings is unusual except in very high narrow buildings. Methods of analyzing such wind stresses are found in our engineering literature.* The problem of determining the bending moment due to unbalanced loads is perhaps the most difficult of the four mentioned. The method of solution used in this paper is by analogy to the behavior of elastic rigid frames of proportions similar to those of the building structure in question. The results hence are not exact solutions of the building frame, but should serve as an indication of the magnitude and distribution of the moments that exist. The analysis should also serve to show the effect of a wide variety of conditions which cannot usually be realized in experimental researches.

METHODS OF ANALYSIS.

The analysis of statically indeterminate frames may be effected by the use of various principles, which may involve the elastic deformations of the members or the internal energy stored up in the structure. From a consideration of the methods employed in solving frame problems, it is seen that such analysis is greatly simplified by conditions of symmetry of structure and load. A similar simplification is possible when lateral movements of the joints of a frame are prevented or where such movements are small enough to be negligible. In the reinforced-concrete building several bays in width, it is usually safe to assume that there is no lateral movement of the ends of columns at each floor level due to the effect of unbalanced loads. For such conditions a modification of the "slope-deflection" method is quite useful. The method consists in reducing the portion of the structure to be studied to one of a few standard types, for which moment equations have been formulated and are listed in Table 1.†

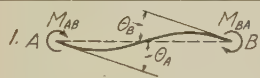
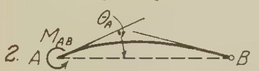
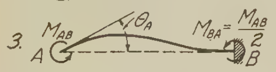
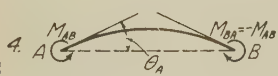
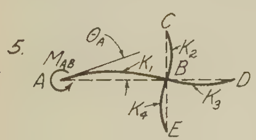
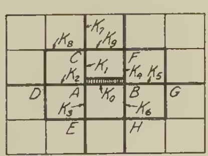
The equations for the typical cases of Table 1 are of such general application that they make it possible to solve many forms of frames by inspection. The significance of the different types of member and equation may be seen from a study of the table. As noted in the table, it is assumed that the ends of members do not move from their original position, except that the slope of the members may change. Case 1 shows a member acted

---

*See Wind Stresses in the Steel Frames of Office Buildings, by W. M. Wilson and G. A. Maney, Bulletin 80, Engineering Experiment Station, Univ. of Ill. Also Engineering and Contracting, V. 53, No. 12, p. 314.

†The method was developed and used by the writer in a master's thesis in 1915; later it was extended in Bulletin 108, Engineering Experiment Station, Univ. of Ill. 1918, and in articles in Engineering and Contracting, Vol. 52, No. 4 and 8, 1920.

Table I.—Moment Equations for Standard Types of Members and Frames.

Type of Member or Frame	*Moment Equations*
Ends of members remain in their original position	*E = Modulus of Elasticity of Material.* *K = Ratio of Moment of Inertia to length of Member.* *θ = Slope of Elastic Curve at End of Member.* *Clockwise moments are considered positive.* *Slope indicating clockwise rotation is positive.*
	General Case $M_{AB} = NEK\theta_A$ When $\frac{M_{BA}}{M_{AB}}$ or $\frac{\theta_B}{\theta_A}$ is known, $N = \frac{6}{2 - \frac{M_{BA}}{M_{AB}}} = 4 + \frac{2\theta_B}{\theta_A}$
	End B Hinged $M_{AB} = 3EK\theta_A$ $(N = 3)$
	End B Fixed $M_{AB} = 4EK\theta_A$ $(N = 4)$
	Ends Symmetrical $M_{AB} = 2EK\theta_A$ $(N = 2)$
	Partial Restraint at B $M_{AB} = NEK\theta_A$ where $N = 4\left[\frac{N_2K_2 + N_3K_3 + N_4K_4 + 3K_1}{N_2K_2 + N_3K_3 + N_4K_4 + 4K_1}\right]$
6-Member AB uniformly loaded. Unequal restraints at A and B. 	General Case $M_{AB} = \frac{wl^2}{12}\left[\frac{N_1K_1 + N_2K_2 + N_3K_3}{N_1K_1 + N_2K_2 + N_3K_3 + (4 - 2\mu)K_0}\right]$ $M_{BA} = -\mu M_{AB}\left[\frac{N_4K_4 + N_5K_5 + N_6K_6}{N_1K_1 + N_2K_2 + N_3K_3}\right]$ $\mu = -\frac{\theta_B}{\theta_A} = \left[\frac{N_1K_1 + N_2K_2 + N_3K_3 + 6K_0}{N_4K_4 + N_5K_5 + N_6K_6 + 6K_0}\right]$ M_{AB} is divided into M_{AC}, M_{AD}, and M_{AE}, in proportion to the respective values of N_1K_1, N_2K_2 & N_3K_3 $M_{CA} = \frac{M_{AC}}{2}\left[\frac{N_7K_7 + N_8K_8 + N_9K_9}{N_7K_7 + N_8K_8 + N_9K_9 + 3K_1}\right]$, etc. $N_1 = 4\left[\frac{N_7K_7 + N_8K_8 + N_9K_9 + 3K_1}{N_7K_7 + N_8K_8 + N_9K_9 + 4K_1}\right]$, etc. Symmetrical Case Frame symmetrical about vertical center line In equations above $\begin{cases} \mu = 1 \\ M_{BA} = -M_{AB} \end{cases}$

upon by the moments M_{AB} and M_{BA} at the ends. The slope of the elastic curve at A is θ_A and that at B is θ_B. If the relation between the two moments, or between the slopes of the elastic curve at the two ends is known, the moment M_{AB} may be expressed in terms of the slope θ_A, the stiffness factor EK of the member, and the restraint factor N.

$$M_{AB} = NEK\theta_A$$

Case 2 follows directly from Case 1, when the restraint of the end B is that of a hinged end; similarly Case 3 applies when B is a fixed end, and Case 4 when the moment at B is made equal, and opposite in sense to that at A. Case 5 represents a member acted upon at A by a moment, and partially restrained at B by the three members BC, BD and BE into which it frames. It is therefore between the conditions of restraint of Case 2 and 3, and hence the value of N may be expected to be between 3 and 4. The expression for N_1 for this case is built up from the equations of the preceding cases; thus if CB is hinged at C then N_2 is 3, if BD is fixed at D,

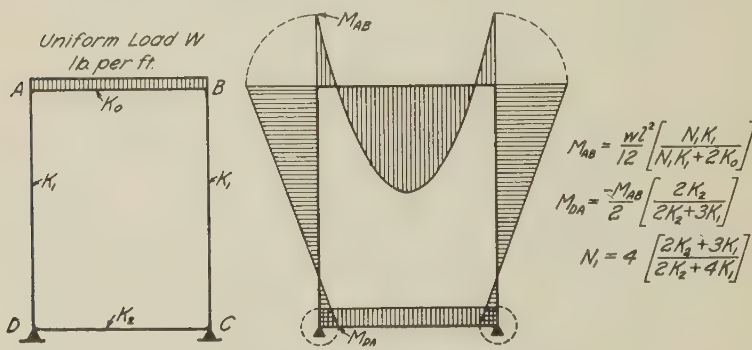


FIG. 1.—MOMENTS IN A RECTANGULAR FRAME.

N_3 is 4. By applying the equation of Case 5 successfully to the adjacent members of a frame, their restraining influence upon a particular member may be found. This has been done in deriving the equations of Case 6. Here the loaded member AB frames into adjoining members at each end. The loading is taken as symmetrical about midspan of AB, but the restraints at the two ends, A and B, need not be equal. It is obvious that any of the restraining members at either end may be dropped out by simply omitting the corresponding value of NK in the equation, or other members may be added at the joint by adding the corresponding values of NK. By determining the restraint produced at the points C, D, E, F, G and H by the members meeting at these points, using the equations of Case 5, the influence of the load on AB may be traced to all of the members of the surrounding frame-work.

To illustrate the adaptability of these standard types of frame and their accompanying equations, the simple rectangular frame of Fig. 1 may

be considered. Here the loaded member AB corresponds to AB of Case 6, Table 1; the member AD to AE of Case 6; and the members AC and AD of Case 6 are simply omitted. The rectangular frame is symmetrical and the moments are given by the equations in Fig. 1. The members AD and DC may be compared with members AB and BC of Case 5, Table 1, whence the value of N_1 is found. The remaining moments, shears, and reactions of the frame of Fig. 1 may now be found by the use of the usual equations of statics.

The equations of Table 1 are particularly applicable to building frames in which there is unbalanced loading. The problems of most interest to engineers are those relating to the moments in exterior columns, although it is possible to develop moments of considerable magnitude in interior columns through unbalanced loading of panels.*

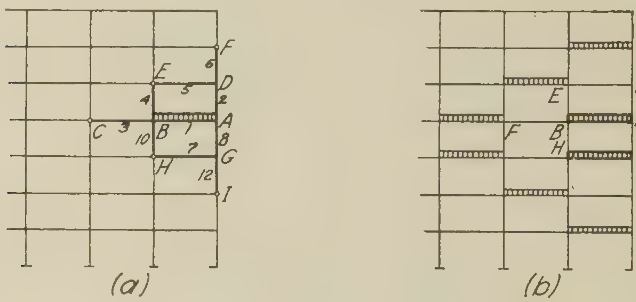
EXTERIOR OR WALL COLUMNS.

It is obvious that the wall column must always be subject to an unbalanced floor load, and that this will usually consist of both dead and live load. If Fig. 2 (a) represents the skeleton framework of a building, the effect of a load on a single exterior panel may be found by considering the portion of the frame shown by heavy lines. A larger portion of the frame could be considered, but the resulting equations are unwieldly. Assuming hinged ends at the extremities of the frame considered, the equations given in Fig. 2 are written by inspection of the equations of Case 6, Table 1. The equations shown are those applying to the exterior column; similar ones may be written for the other members of the frame.

The equations of Fig. 2 may be used to find the effect produced on the column moments by loading several adjacent panels. It is found that the loading producing the maximum moment in the column just below the point A and above the point G is that shown in Fig. 2 (b). More than 90 per cent of this moment is produced by the loads on the two adjacent spans AB and GH; furthermore this loading of the two spans is much more likely to occur than that of the arrangement shown in Fig. 2 (b). Therefore the case of a live load on the two spans AB and GH only has been used in the following studies. Considering first that the value of K (ratio of moment of inertia to length) for all columns may be taken as a constant times the value of K for the girders, the diagram of Fig. 3 (a) has been made up. The diagram gives values of the moment in the column and in the adjacent girder for varying degrees of stiffness of the columns. It is seen that where the ratio of K of columns to K of girders exceeds 2, as is usually the case in concrete building construction and particularly in flat slab construction, that the moment in the column is about 0.6 of the full negative moment.

*Unbalanced loadings on flat slab structures were considered by Prof. H. M. Westergaard and W. A. Slater in a paper "Moments and Stresses in Slabs," Proc. A. C. I. 1921, with particular reference to the effect on the moments in the slab, although moments in columns due to certain loadings were also considered.

The moments in the girder and column when all panels are loaded, as, for example, with dead loads, have been computed from the equations of Fig. 2 and similar equations and are given in Fig. 3 (b). It is seen that the maximum moment in the exterior column is about three-fourths as great for this loading as it is with the load on the two spans only. Moreover, this moment occurs at the top and bottom of the column in each story. The curve for the moment in the girders shows that the degree of



Equations for Frame (a):

$$M_{AD} = -\frac{wl^2}{12}\left[\frac{N_2K_2}{N_2K_2+N_8K_8+(4-2\mu)K_1}\right] \qquad M_{GA} = \frac{M_{AG}}{2}\left[\frac{K_7+K_{12}}{K_7+K_{12}+K_8}\right]$$

$$M_{AG} = -\frac{wl^2}{12}\left[\frac{N_8K_8}{N_2K_2+N_8K_8+(4-2\mu)K_1}\right] \qquad M_{GI} = \frac{-M_{AG}}{2}\left[\frac{K_{12}}{K_7+K_{12}+K_8}\right]$$

$$N_2 = 4\left[\frac{3K_2+3K_5+3K_6}{4K_2+3K_5+3K_6}\right] \qquad M_{DA} = \frac{M_{AD}}{2}\left[\frac{K_5+K_6}{K_5+K_6+K_2}\right]$$

$$N_8 = 4\left[\frac{3K_8+3K_7+3K_{12}}{4K_8+3K_7+3K_{12}}\right] \qquad M_{DF} = \frac{-M_{AD}}{2}\left[\frac{K_6}{K_5+K_6+K_2}\right]$$

$$\mu = \left[\frac{N_2K_2+N_8K_8+6K_1}{3K_4+3K_3+3K_{10}+6K_1}\right]$$

FIG. 2.—MOMENT EQUATION FOR EXTERIOR COLUMNS, AND LOADING FOR MAXIMUM MOMENT.

fixedness is high, approaching the conditions of full negative moment which exist at the ends of a fixed beam. It is also seen that the moment in the girder does not vary greatly whether all spans or only two spans are loaded.

In the preceding analysis it has been assumed that the building frames discussed were made up of columns and girders of uniform length and section. This is a fair assumption with respect to the girders; in the columns, although the story height may not vary appreciably, the section

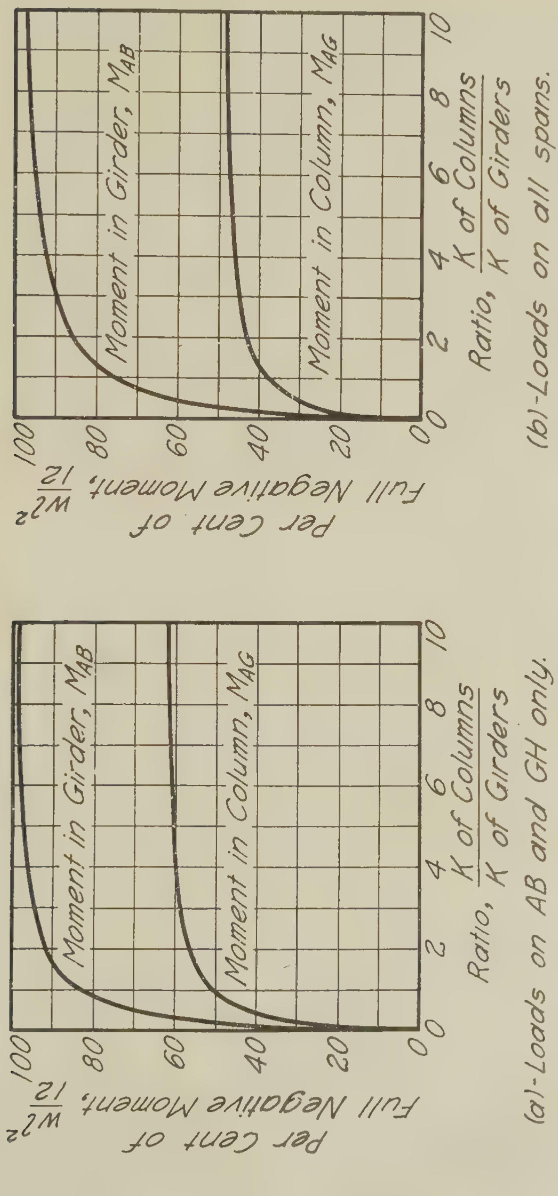


FIG. 3.—MOMENTS IN EXTERIOR COLUMNS OF THE BUILDING FRAME OF FIG. 2 (B).

commonly decreases from the lower to the upper stories, as governed by axial loads. Hence at a floor level where the column section changes, the moment of inertia of the upper column may be considerably less than that of the column below, since with a constant percentage of steel the moment of inertia varies as the fourth power of the diameter of the column. The ratio of the value of K (moment of inertia divided by length) for the upper column to that of the lower one will be denoted by r: values of this ratio have been calculated by reference to the column schedules of existing buildings, and are found to vary from 0.5 to 1.0 with many cases of 0.6 and 0.7. Since the members of a frame take stress in proportion to their relative stiffness, or roughly to their load carrying ability, it is clear that the larger of two columns will receive the greater share of the unbalanced negative moment brought in by a girder. Using several values of the ratio r, values of the moments in the two columns have been found by the methods used in connection with Fig. 3, and curves for these moments are given in Fig. 4 (a). Referring to Fig. 2 (b) with loads on AB and HG only, the average value of K for the columns AG and AD and that of the girders AB and BF have been used to determine the ratio of K of columns to K of girders. An empirical equation has been found to express the relation between the various curves of the diagram. It is found that the moment M_{AB} in the girder is distributed between the columns AG and AD very nearly as indicated by the following equations:

$$M_{AG} = -M_{AB}\frac{4+r}{4+4r} \tag{1}$$

$$M_{AD} = -M_{AB}\frac{3r}{4+4r} \tag{2}$$

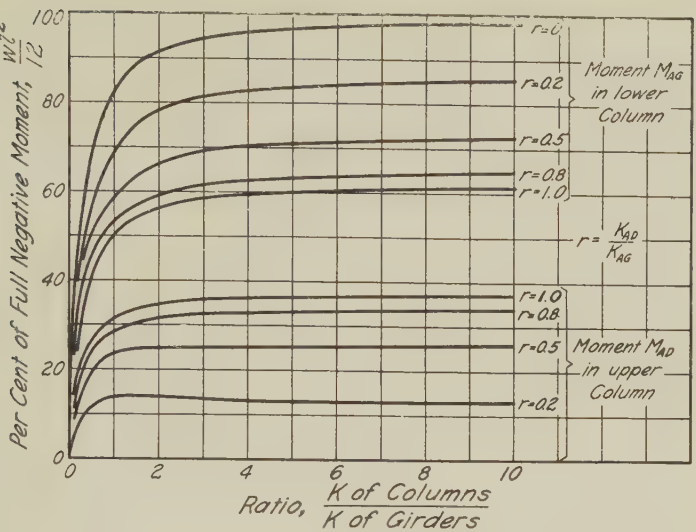
In a similar way when all spans of the structure are loaded, calculations were made to determine the value of the moment in the girder M_{AB}, as shown in Fig. 3 (b). This moment is divided between the columns above and below each story as indicated by Fig. 4 (b), and by the following equations:

$$M_{AG} = -M_{AB}\frac{1}{1+r} \tag{3}$$

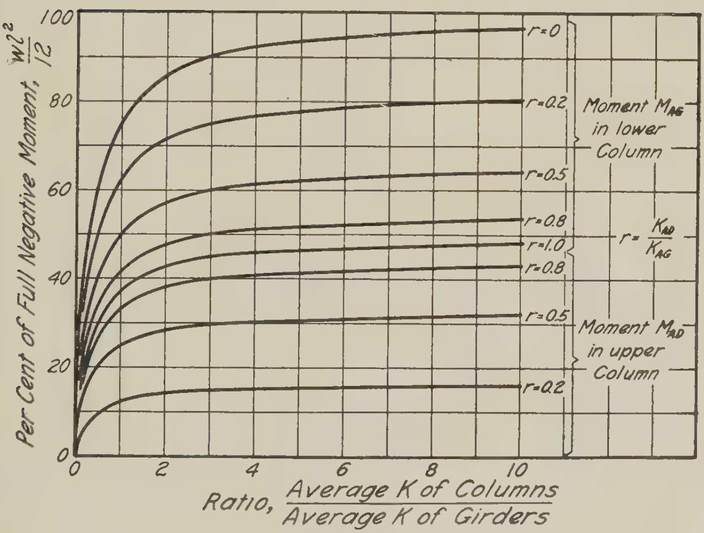
$$M_{AD} = -M_{AB}\frac{r}{1+r} \tag{4}$$

It is apparent that with a value of r of perhaps 0.7, the moment at the top of the heavier column may be 10 and 18 per cent greater, respectively, for the two methods of loading, than the moment existing in columns having a constant value of K. The moment in the lighter column is, of course, decreased a corresponding amount.

The moment in a column at the roof level can be determined quite closely by the procedure of the preceding paragraph. Thus in taking the average of K for the column above and below the roof level, since there is no column above, the average value is obviously one-half of the value of K for the column below. Using this average value, the amount of negative



(a)-Load on AB and GH only.



(b)-Load on all Spans.

FIG. 4.—EFFECT OF VARIATION IN SECTION ON MOMENTS IN EXTERIOR COLUMNS OF THE BUILDING FRAME OF FIG. 2 (B).

moment in the girder, M_{AB} is taken from the diagrams of Fig. 3, and all of this moment is transmitted to the column underneath, as would be indicated by equations 1 and 3 with $r = 0$. The coefficient of moment in roof columns is generally greater than at other floor levels, for while the single column does not offer as great an end restraint and hence does not attract as great a proportion of the full negative moment as would two columns, still this moment must all be carried by the one column. The high moment coefficient, however, is compensated by the fact that the live load on the roof is ordinarily light.

Another variation in column sections is found in the column capitals used in flat slab construction. The effect is similar to that produced by changes in column section; in each case the column just below a floor level is made stiffer than that in the story above, and consequently receives a larger share of the unbalanced moment brought in at the floor level. While the effect of a capital is to increase the stiffness of a column over a small portion of its length, and is not directly comparable with a uniform increase in section of the entire column, for purposes of analysis the column with a capital may be considered to be replaced by an "equivalent" uniform column and the moment distribution in the "equivalent" column then treated through the use of the factor r. For exterior columns the half-capitals or brackets used have less effect than would the capitals of interior columns. By considering a number of wall columns with capitals of typical sizes the moment of inertia of an equivalent column has been found on two different assumptions, (1) Far end of column hinged; equivalent column to produce same slope and same moment at the top as the column with capital; (2) Far end of column restrained, with same slope at both ends of column as produced by the loading of all spans of the structure. A number of numerical cases were treated by both of these assumptions, and the two conditions of a capital on the near end and the far end of a column were studied. In both cases, using a square column with a pyramidal half-capital, the ratio of moment of inertia of the equivalent upper column to lower one was found to vary from 0.77 to 0.84. Table 2 shows the results of calculations made on the second assumption, applying to the case in which all spans of the structure are loaded. This value of r found, indicates in an approximate way that the effect of a column cap is similar in magnitude to the effect of a small increase in section of the lower column at a floor level.

Table 2 also shows the position of the point of contraflexure in the columns of a structure in which spans and story heights are of constant dimensions and in which all spans are loaded. It is noteworthy that the point of contraflexure for the three structures treated is very nearly midway between the top of the floor slab at the bottom of the column, and the bottom of the capital. This indicates the presence of equal moments at the top and bottom of the uniform portion of the column.

For the common condition in which the column below a floor level has both a capital and a large diameter than the column above, the combined

effect on the moment distribution is found by determining the two values of r and multiplying them together. Thus if the value of K for the upper column is 0.8 of that for the lower one, and if the effect of the capital also results in an equivalent lower column for which $r = 0.8$, the combined effect is to make $r = 0.64$, which can be used in equation 1 to 4.

In addition to the information given in Table 2 regarding the position of the point of contraflexure in the columns, the effect of various values of r and various capital heights may be noted. By the use of equations (3) and (4), for the case of load on all spans, the moments at top and bottom

TABLE II.—EFFECT OF COLUMN CAPITALS ON STIFFNESS OF COLUMNS.

Example No	1	2	3
Length of span	l	l	l
Width of column capital	$0.2l$	$0.225l$	$0.25l$
Diameter of column	$0.083l$	$0.10l$	$0.125l$
Thickness of slab	$0.033l$	$0.037l$	$0.04l$
Clear height of capital	$0.067l$	$0.07l$	$0.08l$
Story height	$0.7l$	$0.6l$	$0.5l$

SQUARE EXTERIOR COLUMN WITH HALF CAPITAL.

Moments of inertia, in millionths of l^4	Column section, $I=\frac{d^4}{12}$	3.94	8.33	20.30
	Equivalent column, moment at top	4.71	10.38	26.95
	Equivalent column, moment at bottom	3.97	8.43	20.78
Value of r		0.84	0.81	0.77
Height to point of contraflexure, proportion of clear height		0.49	0.49	0.50

SQUARE INTERIOR COLUMN WITH FULL CAPITAL.

Moments of inertia, in millionths of l^4	Column section, $I=\frac{d^4}{12}$	3.94	8.33	20.30
	Equivalent column, moment at top	4.86	10.80	28.30
	Equivalent column, moment at bottom	3.99	8.47	20 86
Value of r		0.82	0.78	0.74
Height to point of contraflexure, proportion of clear height		0.49	0.48	0.48

of a column and also at the bottom of the column capital have been determined. These moments, shown in Fig. 5, are expressed as a fraction of the unbalanced negative moment M_{AB} in the girder. It is to be noted that while the moment at the top of the column increases with decreasing values of r, the moment at the bottom of the capital, for the two conditions of capital height used, does not exceed 0.5 M_{AB} for values of r greater than 0.5 to 0.6. Hence it may be concluded, that while a column capital will attract a large proportion of the unbalanced negative moment, the moment at the top and bottom of the column proper will not usually be appreciably greater than if no capital was present.

INTERIOR COLUMNS.

Bending moments in interior columns are not generally of particular importance, since if adjacent spans are equal and all spans are loaded, the floor loads are balanced and there is no moment produced in the columns. In the case of equal spans, the dead loal is always balanced, and it is

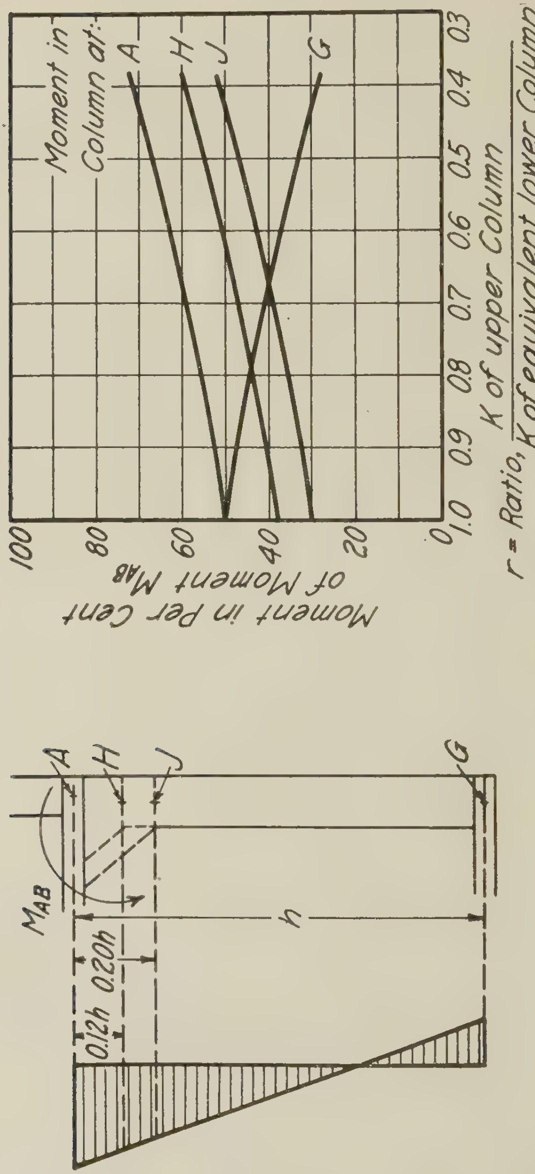


FIG 5.—DISTRIBUTION OF MOMENTS IN COLUMNS HAVING CAPITALS, LOAD ON ALL SPANS OF STRUCTURE.

largely balanced with the usual unequal spans. In considering the effect of live loads, an interior frame similar to that of Fig. 2 (a) has been analyzed, and the effect has been found of loading the different panels of the frame. It is found that the loading indicated in Fig. 6 (a) produces the maximum moment in the column just below A or above G. It will be seen that the arrangement of the loaded panels is quite similar to that of Fig. 2 (b) for exterior columns, and likewise it is found that about 80 per cent of the moment is produced by loads on the two spans AB and GH. Hence, since the latter loading is one that may reasonably be expected to occur frequently, its effect will be studied.

Again considering the building frame of Fig. 6 (a) in which the value of K for columns is taken as a constant times that for the girders, the curve of Fig. 6 (b) shows values of the moment produced in the column at A or G due to loads on spans AB and GH. It will be noted that where the columns are much stiffer than the girders the induced moment in the column approaches that found for exterior columns. The negative moment in the girder at A is distributed among the two adjacent columns and the adjoining girder; hence for the part of the curve where columns and girders are more nearly equal, the adjacent girder receives a considerable share of the unbalanced moment and the columns receive relatively less.

Allowance may be made here for the effect of changes in section of columns and of the use of column capitals by a method similar to that used with exterior columns. By the analysis of a number of frames in which the values of K of the columns of each succeeding upper story were decreased by a constant percentage, the effect upon the moment M_{AG} in the column was found and an empirical equation fitted to the results. To find the value of the moment in the column, M_{AG}, when the ratio of K of a column to the story below is r (a quantity less than 1.0), the average value of K for columns AG and AD and of girders AB and AC is first used to find a value of M_{AG} from Fig. 6 (a), and this value of M_{AG} is multiplied by the quantity $\frac{5+r}{3+3r}$ to obtain the corrected value of M_{AG} in the column at A. Thus it is seen that if $r = 0.7$, the moment M_{AG} is increased by about 12 per cent, and the moment above, M_{AD}, is correspondingly decreased.

Allowance for the effect of column capitals is also made, as was done with exterior columns, by replacing the column and capital by an equivalent column of uniform section, for which a value of r may be computed. The effect of an interior capital depends of course upon the relative proportions of capital and of column; a range of probable conditions has been analyzed and the corresponding values of r are given in Table 2. The results with the full capital do not differ greatly from those for exterior columns in which a half-capital was used. The value of r is found to vary from 0.74 to 0.82, and the point of contraflexure, with all spans loaded, is found to lie very nearly midway of the clear height of the column proper. The observations made from consideration of Fig. 5, regarding the magnitude of the moment at the bottom of the column capital also apply here.

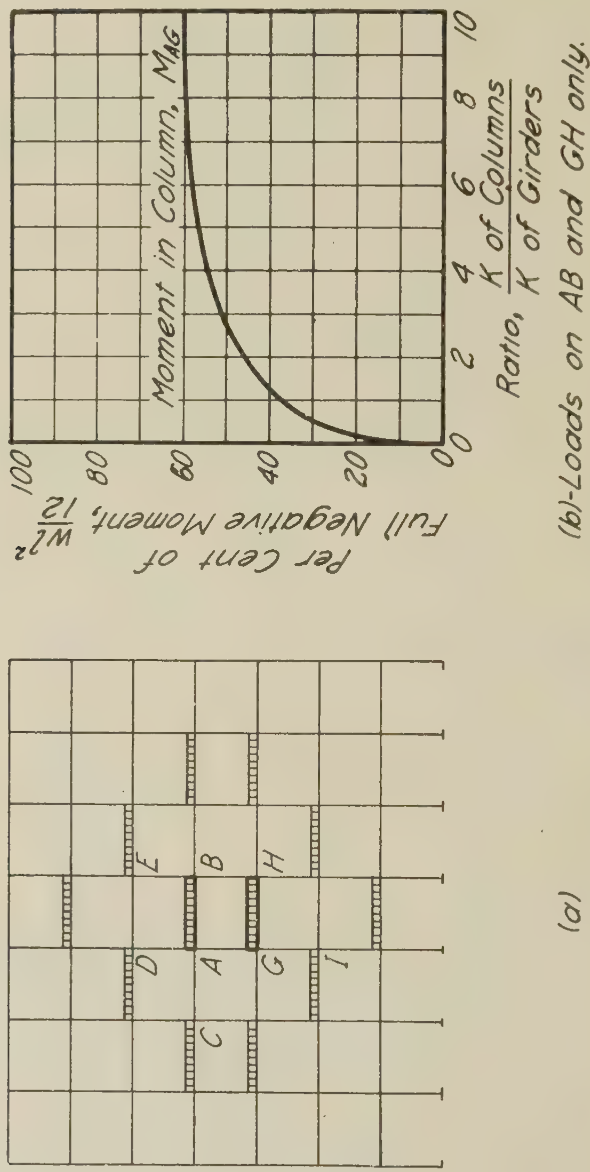


FIG. 6.—MOMENTS IN INTERIOR COLUMNS.

NUMERICAL EXAMPLES OF MOMENTS IN A BUILDING FRAME.

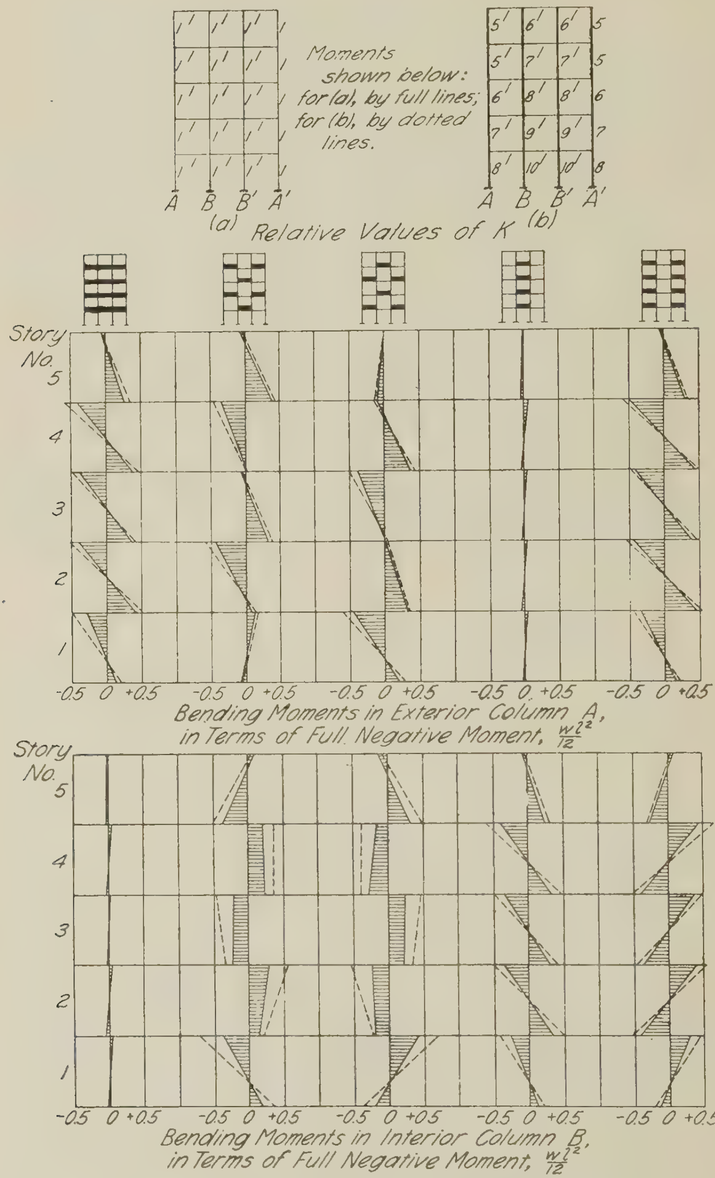
In addition to the foregoing studies of column moments, analysis was made of the moments throughout an entire building bent under different types of loading. A building frame five stories high and three bays wide was analyzed by the exact application of the slope-deflection method, which involved the solution of ten simultaneous equations for each type of loading. The moments found in both exterior and interior columns are plotted in Fig. 7. The analysis was made on two assumptions: (a) that the value of K for all columns and girders was the same, and (b) that the average value of K for columns was several times that for girders, with K for the lower columns greater than for the upper ones. Values of K for both assumptions are given in Fig. 7. The loadings, as indicated in the figure, consisted either of a uniform load on all spans, or of loads on alternate spans and stories, or on alternate bays.

A study of the moment diagrams for both exterior and interior columns shows that for the loadings used there are few cases where the moments exceed $0.6 \frac{wl^2}{12}$, and in most cases they are not more than $0.5 \frac{wl^2}{12}$. From the foregoing studies it is evident that the loadings are not the loadings that produce a maximum moment in the columns. The diagrams of Fig. 7 are useful mainly in giving a graphical representation of the variation in moments throughout the height of a column of a building, and show quite clearly the effect of conditions at the footing and roof levels.

MOMENTS IN FLAT SLAB CONSTRUCTION.

In applying the results of frame analysis to flat slab structures, several adaptations of the conditions must be made. The length of frame members has been taken as the distance center-to-center of joints, and the moments of inertia of members had been considered as constant. The effect of column capitals has already been treated with regard to the moment distribution in columns; their influence upon the amount of negative moment in the slab must also be noted. Figs. 3, 4 and 6 refer to the full negative moment in a girder or slab; in a girder with uniform load this moment is $\frac{wl^2}{12}$ but in a flat slab moment is not quite so definite. The numerical sum of positive and negative moments in a square flat slab panel is theoretically equal to $\frac{wl}{8}\left[l - \frac{2c}{3}\right]^2$, wherein l is the span and c is the diameter of the column capital.* This theoretical value has been reduced in most building codes because of considerations of the existence of tension in the concrete, of the redistribution of stresses at critical sections, of the effect of biaxial stresses, and other favorable conditions of the slab structure. For example, the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete in its progress report in

*See paper and discussion "Statical Limitations upon the Steel Requirements in Reinforced-Concrete Flat Slab Floors," by J. R. Nichols, Trans. A. S. C. E., Vol. LXXVII.


Moments shown below: for (a), by full lines; for (b), by dotted lines.
A B B′ A′
(a) Relative Values of K (b)
Story No.
5
4
3
2
1
-0.5 0 +0.5
Bending Moments in Exterior Column A, in Terms of Full Negative Moment, $\frac{wl^2}{12}$
Bending Moments in Interior Column B, in Terms of Full Negative Moment, $\frac{wl^2}{12}$

1921 used a value 72 per cent of the above theoretical moment, or 0.09 $wl\left[l-\frac{2c}{3}\right]^2$, and took the total negative moment to be from 60 to 65 per cent of this amount. While all of the arguments for using slab moments which are 72 per cent of the theoretical moment do not apply to the action that produces moment in columns, it might be considered inconsistent to specify larger moments in columns than would be provided for in the adjacent slab. Hence it seems proper to use the value of five-eighths of the total moment, or 0.056 $wl\left[l-\frac{2c}{3}\right]^2$, as the full negative moment in flat slab structures, just as the value of $\frac{wl^2}{12}$ may be taken in column-and-girder frames under uniform load.

It may be noted that with a loading other than uniform load on a girder, the term $\frac{wl^2}{12}$ will be replaced by the moment occurring at the end of a fixed beam under the given loading. If the loads are symmetrical about midspan, the "fixed-beam" moment may be expressed as the average ordinate of the moment diagram for a simple beam of like span and loading.

MOMENTS OF INERTIA.

The preceding analyses have made use of the moments of inertia of the cross-sections of the various members of a structure. The moment of inertia of a reinforced concrete member is not a quantity that is frequently used in ordinary design. The combination of two materials obviously requires an allowance for their difference in stiffness; this is usually accomplished by replacing the steel area by an equivalent area of concrete. This is done by multiplying the steel area by the ratio of the moduli of elasticity of steel and concrete. Furthermore there is some question as to whether the moment of inertia of the tension zone of the concrete section should be used. Tests of reinforced-concrete frames made by the writer lead to the following conclusions regarding the moments of inertia of the members as used in determining moment distribution:

1. At loads somewhat below ordinary working stresses, calculations made using the moment of inertia of the full concrete section plus the transformed steel section agreed well with the actual stresses and moment distribution. As the maximum load was approached, regions of high tensile stress developed numerous cracks and the neutral axis shifted toward the compression face of the member. Evidently in finding the moment of inertia at this stage the concrete tension zone should be omitted; even this does not allow for the curvilinear variation of compressive stresses in the concrete, since the use of the moment of inertia of a section presupposes the existence of a straight-line stress-strain diagram. A compensating feature of the decrease in moment of inertia at high loads is that it usually occurs at regions of both high positive and high negative moment, which tends to neutralize the effect insofar as the distribution of moments and shears is concerned.

2. For building frames wherein the columns are subject to high direct compressive stresses that exceed the maximum tensile bending stresses, there will be no cracking of the concrete and the moment of inertia of the full concrete area of the columns should be considered.

3. For purposes of design, since only the relative values of the moments of inertia of the members of a frame are usually desired, the moments of inertia of the full concrete sections (neglecting the steel areas) will give a reasonably close approximation to the actual conditions.

4. For building frames in which the girders have tee-flanges composed of the adjacent floor slabs, these flanges will be useful only in regions of positive moment and it seems reasonable to consider an effective width of tee as not more than that used in stress analysis, such as one-fourth of the span, or twelves times the slab thickness plus the width of the web of the girder.

5. In flat slab construction the effective width of the slab to be used in calculating the moment of inertia would appear to be between 0.5 l and l, where l is the panel length at right angles to the span considered. A study of moment coefficients in slabs shows that the strip running from column to column, of width 0.5 l, differs considerably in action from the middle strip down the middle of the panel. In either two- or four-way reinforcement the column-strip forms a stiff beam carrying a relatively large part of the load. Further, considering that the column will usually have a much higher percentage of reinforcement than the slab, and that the moment of inertia of the slab will be reduced by tensile failure of the concrete, it seems reasonable to limit the width of the slab to 0.5 l in calculating its moment of inertia. The full concrete area (without steel) may then be used for both columns and slabs without serious error.

LOADINGS PRODUCING MAXIMUM STRESS IN COLUMNS.

(a) *Exterior Columns.* Columns are usually proportioned to carry a direct load, due to loads on all adjacent spans of the structure. In exterior columns, there occurs simultaneously a bending stress which when combined with the direct stress may produce a condition of maximum stress in the column. It is possible to produce a somewhat larger bending stress by loading selected panels only, but this will necessarily result in a reduction in the direct compression, and the former condition of loading will probably be the more severe. It has been shown that if the full negative moment in a frame is $\frac{wl^2}{12}$, the moment in columns with all spans loaded is above 0.45 $\frac{wl^2}{12}$ for both live and dead load, while if a few selected spans are loaded, the moment is about 0.60 $\frac{wl^2}{12}$ for live load and 0.45 $\frac{wl^2}{12}$ for dead load. For the upper stories of a building, in which the direct load on the columns is relatively small, the bending stresses are of greater importance than in the lower stories where direct loads are much

longer. Hence, a loading of selected panels will often produce a maximum stress in columns of such upper stories.

(b) *Corner Columns.* The corner column, which carries direct load from only a quarter-panel of floor and from lintel beams, is usually of light cross-section. However, the bending moments produced by unbalanced floor loads are approximately equal to the moment due to a half-panel load, and these moments are applied in two directions on the column. As a resultant of the two moments in the directions of the adjacent edges of the slab, a single moment acting along a diagonal of the corner panel might be considered. The fiber stress at the inner and outer corners of the column is thus equal to that produced in the faces of an exterior column of similar size by a full panel load. In addition there will usually be a bending stress due to loads on the two lintel beams which act in two directions on the column to produce a similar bending in the diagonal direction. Tests of buildings have shown very high stresses in the steel of diagonal bands near the corner column. It appears that bending stresses will generally be more severe in corner columns than in exterior columns.

The criteria for loading to produce maximum combined stresses are practically the same as for other exterior columns. Loading of the corner panels on two consecutive floors will produce maximum bending stresses in the column, but loading of all spans of the building will usually produce the maximum combined stress.

(c) *Interior Columns.* When all spans of a building are loaded there is practically no bending stress in interior columns. Fig. 6 shows, however, that a moment of about $0.60 \frac{wl^2}{12}$ is produced by a load on two spans only. Hence if downward loads are applied on all spans, and upward loads of equal intensity are applied on two spans only, the moment of $0.60 \frac{wl^2}{12}$ is produced. This loading amounts to a downward loading on all but the two spans indicated; in the lower stories of a building it furnishes the combination of the maximum bending stress, with a large proportion of the maximum direct stress, since the column receives all but one half-panel load of its designed direct load. In the extreme upper stories of the building, the loads on two panels only may produce a combined stress greater than that of full axial load alone, just as with exterior columns.

Considering the plan view of a floor slab, it is possible to choose an arrangement of panel loads to produce bending in two directions in a column, similar to that in corner columns, but the occurrence of such loadings is improbable, and hence their consideration in design does not appear justifiable.

TESTS ON MOMENT DISTRIBUTION IN COLUMNS.

(a) *Tests of Rigid Frames.* Several series of tests have been made in recent years to investigate the action of rigidly connected frames of reinforced-concrete. One series, made at the University of Illinois * consisted of tests of eight frames, representing five different types. The frames ranged from 4 ft. 0 in. to 9 ft. 8 in. in height and from 6 ft. 0 in. to 15 ft. 4 in. in width. Members varied in section from about 8 in. by 8 in. to 8 in. by 18 in. in size. Concentrated loads were applied either at midspan or at the one-third points of the span.

From the tests of these frames it was concluded that the elastic action of the frames and the manner of stress distribution agreed fairly well with mathematical analyses. With properly designed connections of members there was no question as to the continuity of members at joints. The moments transmitted to the columns were as much as 50 per cent of the full negative moment for the girders in some cases. Since the direct compressive loads on the columns were comparatively small, the tensile bending stresses were large and many cracks were produced in the columns. Failure occurred at middle of girders or at junction of girder and columns. Similar results were found in an investigation of continuous beams and frames made in Dresden in 1912.‡ One type of rigid frame was tested. The results of the test agreed very well as to moment and stress distribution, maximum load, and manner of failure with the calculated data. In the calculations, corrections were made for the decrease in moment of inertia of girder sections at midspan as the maximum load was approached.

A third series of tests of reinforced-concrete frames was carried on in 1918 at Bethlehem, Pa.* Nine frames 7 feet high and 15 feet wide, were tested, of which six had fillets or brackets at the junction of columns and girder. In these latter frames the moments induced in the columns were very large (70 to 85 per cent of the full negative moment for the girders), and the frames failed at the junction of columns and brackets.

In all of these tests that have been cited, it is noteworthy that all of these reinforced-concrete frames have proved susceptible of mathematical analysis. The agreement between analysis and tests appears as close as that found in tests of simple reinforced-concrete beams. The rigidity of properly designed joints is unquestioned, and the tests serve to prove that the action of such structures can be analyzed.

(b) *Tests of Buildings.* Very few of the available reports of strain gauge tests of reinforced-concrete buildings contain information regarding the moments and stresses in the columns. Measurements of deformations in columns were made in the tests of the Shredded Wheat Factory build-

---

*Analysis and Tests of Rigidly Connected Reinforced-Concrete Frames, by M. Abe, Bulletin 107, Engineering Experiment Station, Univ. of Ill. 1918.

‡Untersuchungen an durchlaufenden Eisenbetonkonstructionen, by Prof. H. Scheit and Dr. Ing. E. Probst. Berlin, 1912.

*See Structural Laboratory Investigations in Reinforced-Concrete made by Concrete Ship Section, Emergency Fleet Corporation, by W. A. Slater. Proc. A. C. I. 1919, p. 48.

ing,* the Soo Line Freight Terminal slab,* the Schultz Baking Company building,* the Worcester test slab,* and the Western Newspaper Union building.† It is stated in the reports of these tests that the data on deformations in columns are not sufficiently extensive or definite to establish conclusions as to the amount of moment carried by the columns, but in all cases it is shown that quite large bending deformations existed in columns located at edges and corners of loaded areas. In the Shredded Wheat Factory tests, tensile and compressive unit deformations in the concrete of 0.0003 and 0.0004 were measured in columns at the edge of loaded panels at a test load of about one and one-half times the designed live load. In the Soo Terminal Slab, deformations in the slab due to the test load were very low, but unit deformations in the columns reached as high as .0004 and .0008 in both tension and compression, and cracks were formed on the tension side of columns, although there was direct compression present in the columns due to dead load which could not be included in the test measurements. It should be noted that this was a one-story structure and the slab was made exceptionally thick to carry train loads. In the tests of the Schultz Baking Company Building few measurements were made on the columns, the greatest deformations being found at a column at the corner of the loaded area. The same observation was made in the Western Newspaper Union Building, wherein unit deformations of 0.0006 and 0.0007 were found on a column at the corner of the test area at a floor load of 913 lb. per square foot or 3.65 times the designed live load. Stresses measured in column reinforcing bars at the side of the test panel were 8100 and 10500 lb. per sq. in. and fine cracks were observed on the tension side of columns near the bottom of the capital. Column stresses measured in the Worcester test slab were in some cases beyond the yield point of the reinforcement, and brought about local failures in the structure. Readings were taken on wall and corner columns, having both large and small capitals. The large capitals added much more stiffness to the slab than to the columns. As a result one corner column failed before the yield point was reached in the slab reinforcement. Very high bending stresses were found in other columns at the maximum load.

The foregoing examples of observed moments in columns indicate the need of more extensive field tests on columns and the correlation of the results with the measurements on the slab structure so that the moment coefficients for the columns may be formulated.

RESISTANCE OF COLUMNS TO COMBINED STRESS.

Design formulas for members subjected to combined bending and compression are generally derived by superimposing the diagram for uniformly distributed axial stress upon that for bending stress. For homogeneous

*Tests of Reinforced-Concrete Flat Slab Structures, by Arthur N. Talbot and Willis A. Slater, Bulletin No. 84, Engineering Station, Univ. of Ill.

†Test of a Flat Slab Floor of the Western Newspaper Union Building, by A. N. Talbot and H. F. Gonnerman, Bulletin No. 106, Engineering Experiment Station, Univ. of Ill.

materials the result is the familiar equation for stress due to eccentric loads:

$$S = \frac{A}{P} \left[1 \pm \frac{ec}{r^2} \right] \tag{5}$$

where S is the maximum or minimum fiber stress, P is the total load, A is the area of the cross-section, e is the eccentricity of the load, c is the distance from the gravity axis to the extreme fiber, and r is the radius of gyration of the cross-section. Without going into the involved equations which can be found in many reference books, it may be said that the above equation, modified by the replacement of steel area with an equivalent concrete area, applies to the case of combined stresses in reinforced-concrete members.

Tests made by Withey at the University of Wisconsin,* by Bach and Graf in Germany,† and tests made at the University of Illinois on eccentrically loaded columns all show a close agreement between measured stresses and calculated stresses. The German tests were made on square longitudinally reinforced columns with closely spaced ties, the others were made on round columns having longitudinally and spiral reinforcement. It appears that spirals and ties had little effect on the distribution of stress over the cross-section, the deciding factors being the percentage of reinforcement and the quality of the concrete.

From all of these tests it is apparent that the critical point in the action of eccentrically loaded columns is reached when the yield point of the compressive longitudinal reinforcement is passed. Large deflections of the column follow and a redistribution of stress occurs. Specimens with a high percentage of reinforcement carried a maximum load as much as 60 per cent above the yield point of the column, while with smaller amounts of reinforcement the maximum load was only about 20 per cent above the yield point of the columns.

In the German tests, eccentricities of as much as 1.25 times the overall depth of the square column were used. In other tests, eccentricities up to one-sixth of the diameter of the columns were used. It is plainly seen that with very small eccentricities the action of the column resembles that of the axially loaded column, with initial failure due to compression in the concrete. With large eccentricities, the conditions of plain bending are approached, and failure occurs when the steel on the tension side of the column passes the yield point. The tests at Wisconsin and Illinois show that while the stresses in the column beyond its yield point do not follow as closely the values calculated from equation (5) as at lower loads, even the ultimate loads may be calculated by means of equation (5) without an error of more than 10 to 15 per cent. This is found to be true with spirally reinforced columns as well as with tied columns.

---

*Tests of Reinforced-Concrete Columns, by M. O. Withey, Bulletin No. 466, University of Wisconsin, 1911.

†Heft 166 bis 169 der Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, von C. v. Bach and O. Graf.

The tests by Bach and Graf showed that when the eccentricity in the tied columns was such as to produce a high compression in the concrete and high tensile stresses in the steel reinforcement, the ratio of the calculated fiber stress in the concrete to the strength of plain concrete cubes was from 1.20 to 1.25, while a similar ratio for concentrically loaded columns was about 0.77 and for highly reinforced members in plain bending (using the straight-line theory), from about 1.50 to 1.80. Many other tests have shown that the calculated compressive stresses in concrete beams are much greater than the strength of the plain concrete as found from compression tests. That is, a much higher stress can be developed on an extreme fiber in bending, than can be developed when the stress is uniformly distributed over the cross-section. It appears, therefore, that a higher working stress in compression may be used on the extreme fiber of eccentrically loaded columns than may be used as the working stress for concentrically loaded columns.* This statement may require limitations in the case of spirally reinforced columns with a large eccentricity of load. Even with an eccentricity of one-twelfth to one-sixth of the diameter of the column, the stress in spirals varies greatly around the circumference of the column, being greatest at the side of maximum compressive stress; and high bond stresses are developed along the spiral. Furthermore the effectiveness of the spiral may be expected to decrease as the eccentricity is increased, due to lateral contraction in the increased tension zone. Tests of spirally reinforced columns with large eccentricities of load are needed to show whether fiber stresses may be developed which are higher than the stresses developed in axially loaded spirally reinforced columns.

SUMMARY.

In review, it may be stated that while bending moments are not commonly considered in the design of reinforced concrete columns, it is known that such moments exist and frequently cause stresses of considerable importance. Provision for such stresses which are practically certain to occur with ordinary conditions of loading seems logical and proper. At the same time it seems justifiable to use higher working stresses for the condition of an extreme fiber stress than for the condition of an uniformly distributed stress which has no possibility of a favorable redistribution over the cross-section.

The values of column moments given in this paper are not the result of freakish or improbable loadings. There are many other possible arrangements of load that will produce moments nearly as great as the ones given. It is felt that the moments shown may be taken as a basis for the design of structures of ordinary proportions. For other types of structures the methods used here should prove useful in analyzing the effect of the particular conditions encountered.

---

*A study of this subject has been made by Mr. A. R. Lord, in the work of Committee E-1, A. C. I., and recommendations regarding working stresses are being formulated.

DISCUSSION.

Clyde T. Morris.—I wish to express my appreciation of the work Mr. Richart has done in preparing this paper and in getting these column moments into shape where a man can use them in an actual design without the extreme amount of work that is necessary in the slope deflection method, and still get results which he can feel are somewhere within reason. I want to call attention to a property of concrete which may have some bearing on the stress that will be developed at the connection of the column and girders, and that is the property of flow, the time factor in adjusting the concrete to the deformations which take place. The flow of concrete is a very definitely known factor, and Mr. McMillan's exhibit out here in the exhibition room showing the flow in a beam or slab that he has had loaded for ten years is very instructive. The deformation of concrete continued for something like four or five years before the curves became horizontal. I think that the dead-load moment in these connections can perhaps be greatly reduced if not neglected in the design. The live-load moment of course must be taken into account, because it may be there today and away tomorrow. I just wanted to call attention to this fact, and this may be one reason why there are fewer failures than you would naturally expect when you analyze existing buildings according to the slope deflexion method. When you find the theoretical bending moments at the corners run the fibre stresses up to 1800 lb. per sq. in. without failure, I think perhaps flow has a good deal to do with it.

S. C. Hollister.—There may be some interesting viewpoints on the paper when you transcribe some of the moments that exist in a reinforced-concrete building into unit stresses. A couple of years ago I analyzed an eight-story warehouse frame for the purpose of investigating what the possible moment in the columns might be. The live-load was 200 lb. per sq. ft. uniformly distributed. It was found that the unit stresses in the wall columns in the lower stories averaged about 80 lb. per sq. in. on the extreme fiber; that the last story,—not the roofs but the last story,—had stresses of about 800 lb. per sq. in. extreme fiber, and that the roof story had over 1000 lb. per sq. in. extreme fiber stress. Now these stresses would indicate that there would be a very serious condition in the frame structure if we took them on their face value.

Professor Morris has already referred to the flow in concrete under load. We know that in the case of ordinary concrete cylinders and also in the case of beams and slabs tested under continued loading, concrete relieves itself or compacts itself in a plastic fashion under continued loading. The steel stresses are affected somewhat by this condition on the tension side, but if there is any compression of steel reinforcement, the stress in the compression steel is greatly increased. Correspondingly, the compressive stress in the concrete where the compression area is reinforced is considerably reduced.

The likelihood of loading such as have been described by Professor Richart in his paper is an important consideration in the design of such a structure and in taking into account such coefficients as he has shown. It is quite probable that moments of the magnitude he describes will be found in the average structure subject to variable load conditions. This, of course, does not, as a rule, apply to such buildings as office buildings, hospitals, schools, and the like; but it is much more important in the case of heavy duty buildings. The coefficients as described in the paper are a function of the relative stiffness of the columns and the girders, as has been shown in some of the charts, especially in Fig. 3 of the paper, in which the ordinates in the chart are percentages of a full negative moment of the loaded panel and the abscissae are the ratio of the stiffness of the column to the stiffness of the girder. Now it is very important to recall that in the lower stories where the columns are stiff, it may be on a flat portion of these curves; but that, correspondingly, since our structure is heavy at that portion of the frame, the unit stresses resulting from such moment are relatively small. However, in the upper portions of the stories where the unit stresses are high, the ratio of the stiffness of the column to the stiffness of the girder is seldom more than unity; in other words, the moment coefficient is much less than it would be in the case of a column in the lower portion of the frame. That means that a coefficient of 40 per cent of the fixed moment is sufficient for consideration in the upper story; and although one might consider even a higher percentage in the lower stories, it is hardly worth while to make that separate analysis because of the relatively small unit stresses resulting. Starting from the roof down, if one investigates approximately four stories, he has usually covered the maximum effect of these eccentric loadings. Except in special cases, the unbalanced moment in the wall column may be balanced by carrying the spandrel beam on the outside of the wall column and carrying the wall on that spandrel beam.

As Professor Morris has stated, it is quite likely that because of the flow in the concrete of the column and of the frame in general, eccentricities due to dead-load are very likely compensated by the plastic effect. The live-load moment, therefore, is the thing which should be considered largely in the matter of eccentricity.

The flow in the concrete introduces another very serious consideration when we analyze combined flexural and direct stresses. In the case of a spirally reinforced concrete column, the unit stress on the core for axial loads is relatively high, as when compared to the unit stress allowed in the extreme fiber of a beam, for example. If we submit a spirally reinforced-concrete column to a considerable flexural stress so that there is an added increment due to the eccentricity of the load or due to the moment, then on one side of the column we have a very high stress and on the other side a smaller stress. If the flexural stress only is relatively large when compared to the combined flexural and direct stress, undoubtedly we should reduce the combined stresses in the column over what we would normally consider for axial loading.

The hooping of the column is an important function in column carrying capacity. If we have flow of concrete in a column, it has become of greater importance and we cannot further endanger the situation by increasing on one side of such a system the unit stress in the core and reducing it on the other side, thereby causing a serious eccentricity. By virtue of the flow in the concrete, also, the stress in the longitudinal steel has become considerably increased, or as some have offered as a suggestion, the longitudinal bars may slip somewhat and thus relieve some of the stress. Whether there is slipping or actual maintenance of the bond between the concrete and the steel, there is undoubtedly an increase in the stress in the longitudinal steel and a decrease in the stress of the concrete due to flow. In general, so far as the combined stresses are concerned, the serious condition arises only when the flexural stresses due to live-load form 30 per cent or more of the combined flexural and axial stresses of the concrete. So far there are very little data on the subject of flow in columns of reinforced-concrete. There is certainly a very much needed field of research, both for the subject of columns and for the subject of other members such as arches and structures where the compressive stress is continuous over a long period.

W. A. Slater.—I do not want to take up very much time; I want to suggest something which may seem a little bit bolshevik in its nature. The load tests and strain-gauge measurements that have been made indicate pretty clearly when all the elements are taken into account that the moments which have been found by rigid analyses are present in a structure. They indicate that in general the stresses in the columns are more severe than the stresses in the slabs, and yet we do not know of any failures of buildings except during construction, usually during cold weather and where bad construction conditions are combined with design so deficient that it would not meet the requirements of any code. It seems to me that instead of increasing the size of columns, there is a possibility that the adjustment between our analytical results, the results of the tests, and the consideration of economy, may even come to the point of modifying downward the moment coefficients for the floor portions of the structure, somewhat as we have done for flat slabs on the basis of the test results.

The subject of what may be used as the moment of inertia of flat slab has been discussed briefly. The results found in the test of a flat slab at Worcester, Mass., in 1913, reported in Bulletin 84 of the University of Illinois, indicate that for the wall panels of that slab considerably less than half the width of the slab from column to column ought to be used in computing the moment of inertia. At the wall column those bars which fell outside the column capital got very little stress in proportion to those which crossed the column capital. I think the reason for this and its bearing upon the subject under discussion will be obvious. In this slab there was no wall beam.

E. S. Martin (*by letter*).—This question of secondary stresses was quite generally neglected ten years back but is now generally required to be investigated in designing. The method of applying these principles merits more discussion than appears in our transactions. Without so definitely stating, it is the intent apparently of our Building Codes Committee Report that designs shall satisfy secondary and direct stresses in the same way and with the same limiting unit stresses. I submit that this is unnecessary and is not the practice in structural steel design.

A requirement that combined bending and axial stresses shall be within the limits given in our building codes for next sections of fireproofed columns is equivalent to the assumption that a disastrous fire will occur spalling off the 1½ or 2 in. of fireproofing when the column is under the most adverse conditions for flexture and direct stress and that we want it to be perfectly safe with the usual factor of safety. One should bear in mind that we probably have a half million concrete buildings in America, while such disastrous fires that have so far occurred in them can be counted on one's two hands. We propose, then, to design 50,000 buildings so that one of them may have a factor of safety of four in this bad fire. It is really saddening when concrete engineers get into the frame of mind of the old maid political orators tell about, who got to thinking one day that if she should have a beau and if they should fall in love, and if they should get married and if they should have a baby and if the baby should get the whooping cough and die, how bad she would feel.

Gross sections of columns should be considered in investigating bending with axial loads both for determining the moments and unit stresses sustained.

Even after going to the length of providing sections such that combined stresses will be within usual requirements in a disastrous fire with eccentric loads and fireproofing spalled off, verticals probably heated, at least exposed and bond broken, how much flectural strength does a concrete engineer think such a column has?

F. E. Richart (*by letter*).—Replying briefly to these very welcome discussions of this paper, I must agree with Professor Morris that plastic flow of the concrete may relieve a large part of the dead load bending stress in columns. The condition is analogous to that in the continuous reinforced-concrete beam, where it has long been recognized that permanent deformations may occur under dead loads. Here, while this has resulted in the specifying of coefficients for negative moments at the supports, under combined live and dead loads, that are somewhat smaller than the amount indicated by elastic theory, I doubt whether we are ready to accept co-efficients based on a zero negative moment for the dead load portion to be carried. More data are needed on plastic flow of concrete in restrained or indeterminate structures.

Regarding the statement by Mr. Hollister that in the upper stories of a building the ratio of the stiffness of the column to the stiffness of the girder seldom exceeds unity, thus resulting in a column moment smaller

than the ones suggested in the paper, I believe that this may apply to beam and girder construction where the depth of girder is not restricted, but it will not apply in general to flat slab construction. I doubt, too, whether it is safe to say that the upper four stories of a building will contain all columns in which the bending moments need consideration.

The questions of flow in spirally hooped columns and of failure of columns subjected to large eccentricities of load are matters concerning which there may be a great deal of speculation; the only convincing answer to these questions lies in adequate and careful observation and study of columns in field and laboratory.

Objection is made by Mr. Martin to the consideration of what he terms "secondary" stresses combined with axial stresses under the same allowable unit stresses. This may be answered by quoting from the paper; "It appears that a higher working stress in compression may be used on the extreme fiber of eccentrically loaded columns than may be used as the working stress for concentrically loaded columns," with certain limitations in the case of spirally hooped columns.

As methods of stress analysis are improved to meet the actual conditions of the structure, the need for a large factor of safety diminishes. As we all know, the allowable extreme fiber stresses in reinforced-concrete are gradually being revised upward, as are also the allowable shearing unit stresses. Such changes must come slowly and depend upon the quality of materials and workmanship that can be secured regularly as well as upon the degree of refinement used in design.

Question Box for Construction Superintendents.

Vice-President M. M. Upson in the Chair.

Mr. Upson.—These questions before you are not drawn from the imagination of our secretary. They are actual inquiries that have come from the various men who have been solicited for subjects of importance to put before a meeting of this kind. And as we progress with these I hope that anything that is collateral or has a bearing that may come to your mind will be brought to the attention of the meeting.

We will take up the questions in order.

Cold Weather Methods.

Discuss protective methods in mixing and placing concrete in freezing weather with a view to obtaining the best results possible.

. . .

What is the effect of freezing temperature upon plain or reinforced-concrete slabs? What is the effect upon plain walls and reinforced columns and beams (assuming that 7/8-in. form work is used)? To what extent does heating with fires under forms affect the result? What is a safe table to use giving the time necessary to leave supports below beams and slabs under the above weather conditions?

. . .

My chief concern now is concreting in cold weather—how many salamanders per cubic ft. of building and canvassed-enclosed walls. Do all or any of the anti-freezing compounds work satisfactorily under general building conditions and what is the lowest temperature at which concrete can be placed successfully? I appreciate that if one had an unlimited expense account to have everything ideal, concrete can be placed at a very low temperature, but every contractor's aim is to put it in at the least expense, and ordinary buildings, unless the owner will stand the extra expense, do not permit this additional cost.

. . .

How do you keep curtains on a building when the wind blows from 40 to 60 miles per hour? Would really like to know, as I have had 30 curtains blown to shreds this past week.

Accelerators.

I have many times been provoked at cement concrete setting slowly in cold weather; a cement finisher will have to wait several hours for the cement to set sufficiently to work it and of course charge for the time waiting; with the present high wages, it makes

quite a difference between winter work and that of summer. Is there anything a person can use to hurry the setting, without injuring the strength of the finished product, outside of heating materials to be used? The firm I am with is rather suspicious of all kinds of dope advertised for this purpose.

. . .

What are the advantages or disadvantages of using calcium chloride to accelerate the hardening of concrete? If calcium chloride is used in floor finish, does it have any effect on the use of floor hardeners at a later date? Does calcium chloride, when used in concrete have any effect on steel reinforcement or structural steel coming in contact with the concrete? Has any trouble been experienced by cement finishers when an accelerator has been used in floor finish?

J. C. GRADY.—We have just completed a very interesting structural steel job. We estimated the job with a winter weather allowance. Our actual cost today, pursuing a different method, that is, precasting the concrete roof slabs and putting them up in place, is exactly the same as if we had cast it in place and in summer weather.

MR. UPSON.—How do you join them?

MR. GRADY.—They join on the top of the I-beams. We fill the joint with mortar after they have set.

MR. UPSON.—Do you protect that?

MR. GRADY.—We put calcium chloride in that mortar, yes. It probably froze a little, but not enough to hurt anything, just enough to hold them in place.

MR. UPSON.—What is the size of those slabs?

MR. GRADY.—Two feet wide and eight feet long.

MR. UPSON.—How do you handle them?

MR. GRADY.—A little hoist, with about a 15 hp. electric motor, and a little A-frame that slides along and hoists them right up and sets them in place as they go on the roof.

MR. UPSON.—Had anyone had any experience in putting anti-freezing material into the aggregate? I can recall very well 23 years ago building one of the first reinforced-concrete buildings that I ever had anything to do with, where we used a very liberal supply of salt in our water. We placed concrete with the thermometer at 25 below zero, and in comparatively thin walls, walls from 8 to 14 in. thick. This seemed to work very well. We kept thermometers in pockets in the concrete, protected them with straw, and had all our materials well heated, so that the temperature of the mixture when it went into the wall was about 85 deg. The walls turned out remarkably well, no defects whatever.

I would like to ask Professor Abrams if the latest findings of the Lewis Institute are against the use of salt water.

D. A. ABRAMS.—We have made a great many tests on the effect of common salt and other soluble compounds on the strength of concrete. We have not made any tests with reference to their effectiveness in pre-

venting freezing. We found that 2 or 5 per cent of salt had the effect of increasing the 3-day strength of concrete about 20 per cent; with the higher percentages of salt the strength of concrete was in all cases below normal; at test ages after 7 days all percentages of salt decreased the concrete strength. At ages of 28 days and older the strength of concrete mixed with 3 per cent salt was about 82 per cent of normal; 5 per cent salt about 75 per cent of normal; 10 per cent salt about 65 per cent of normal and 20 per cent salt about 60 per cent of normal strength.

Sea water, which contains about 3.5 per cent of salt (mostly common salt) gave about 85 per cent of normal strength for concrete cured in a damp condition, and about normal strength for concrete cured in air. In other words, so far as strength is concerned, sea water gives about as good results as ordinary water. This has also been borne out by experience in the use of sea water in construction work. I have in mind particularly the work on the Florida East Coast Railroad; most of the concrete viaducts in the Key West Extension were built with sea water. So far as I know no tests were made, but the actual behavior of the concrete, which has now been in place some 12 or 15 years, indicates that entirely satisfactory results were secured.

MR. UPSON.—Have you found any substitute for salt as an anti-freezing remedy?

D. A. ABRAMS.—I would be inclined to say as a result of the tests made by ourselves and others that calcium chloride is much more satisfactory than common salt. Salts for use in freezing weather have two very different functions:

(1) To slightly depress the freezing point of the water:
(2) To accelerate the early hardening of the concrete.

The following table shows the effect of certain percentages of common salt and calcium chloride on the freezing point of water:

	Freezing Point—Deg. F.	
Solution by weight	Common Salt	Calcium Chloride
0	32.0	32.0
1	30.5	31.1
2	29.3	30.4
5	25.2	27.7
10	18.7	21.4
20	6.1	—1.5

Commercial calcium chloride was considered (75 per cent $CaCl_2$).

It is seen that too much dependence should not be placed on the lowered freezing point of the mixing water. However, against small degrees of frost, they would be helpful if used with discretion. 5 per cent common salt lowers the freezing point about 6 deg. F., but reduces the compressive strength of the concrete about 25 per cent.

Calcium chloride has a very much greater effect than any other salt we have experimented with in hastening the early setting and hardening of the cement. It seems to behave differently with different cements, and we don't know just why it acts that way at all, but for the usual cement we get an increase in strength at 2 to 3 days ranging from 20 to 40 per cent. At 7 days that effect is a little less, probably from 10 to 15 per cent. At 28 days it has practically vanished, and in later days there is no material effect at all.

A more complete report on our tests of concrete mixed with salt water, sea water, etc. will be found in my paper on "Use of Impure Waters for Mixing Concrete."

For cold-weather concreting, it is much better to depend on heating the materials and the concrete and on keeping the structure warm by the application of heat, than to place too much dependence on chemicals.

Mr. Upson.—Isn't it true that the setting of cement generates heat usually?

D. A. Abrams.—It does always; but the heat is generated so gradually and is dissipated to such an extent that in most cases it is of little importance. In large masses of concrete a very material increase in temperature occurs during the setting of the cement; temperatures above the boiling point have been observed.

W. F. Tubesing.—The only way to protect concrete work in winter is to have all the aggregate properly heated, so hot that there won't be big balls of frozen material in the aggregate, but so that when that concrete comes into the forms, if you stand in it with your rubber boots you can feel the heat coming through your boots. And then have sufficient salamanders. We should put on record just how many salamanders you need for a certain number of column spacings, with a certain height of floors, etc.

J. A. Turner.—We canvas in around the outsides so as to make a tight enclosure. Then we leave heat holes through the slab to permit the heat to come up through the top, and then cover the top surface also with canvas, so you will have the same temperature not only on the bottom of your slab, but also on the top of the slab.

Mr. Upson.—Have you any definite rule with respect to the number of salamanders you use per square foot?

Mr. Turner.—That varies a good deal according to the locality. Taking temperatures like the Middle States average, or New England, you have to have space in salamanders to produce a temperature of somewhere near normal, 60 or 70 deg. I think it is very important, if the weather is cold, that the heat holes be provided and canvas put over the top, so you get the same setting from the top down in the slab as from the bottom up.

Mr. Upson.—How close to the column do you ever put a salamander? What is your minimum distance?

Mr. Turner.—It has to be kept away 3 or 4 ft., so there is no danger of setting fire to the canvas, and the same applies to the canvas and the

shores. It is particularly important also to build coke fires and not wood fires. I have seen times where they put sticks of wood in and the stick will probably stand up quite a ways above the salamander and it will finally burn off at the bottom and the stick will fall over and set fire to something.

Mr. Upson.—I think there is another point there that should be made plain, and that is the danger of heating your concrete and drying it out too quickly. My observation has been that a too rapid and too great heat close to green cement very much deteriorates the cement.

N. J. Gould.—How about this tenting in over the top; how do you keep that out of the concrete? A-frames?

Mr. Turner.—We have a light framework that holds the canvas up a foot or two above.

J. G. Ahlers.—I think there is only one safe rule to follow in the number of salamanders to use, and that is to have thermometers at the top of the columns and down near the bottom, because invariably the columns will freeze at the bottoms first. And then make your night men—in the daytime you always have it warm enough—but have your night men keep records of those temperatures. That is the only way I have found it safe to know what is going on where you are not there at night. One rule we follow is one salamander per bay per floor and increase that when it is cold or windy and keep plenty of salamanders on your windward side.

O. F. Priester.—We have been heating our sand and using rather warm water, but not heating the gravel aggregate up, and we have found that we maintain a temperature of about 60 to 70 deg. in the concrete as it goes into the forms. I wonder if the men who in addition to the above heat their gravel, find their concrete as it goes into the forms can be maintained at that temperature.

Instead of covering with canvas, we have covered over the thinner slab with a layer of straw, about 18 in. thick, where we haven't finished floors.

Mr. Upson.—Do you leave holes through?

Mr. Priester.—No, we haven't done that. We have found that the concrete at the top is set, and that we get a hard surface.

C. G. Dunnells.—On several operations in Pittsburgh, in the downtown part of the city where the steam was easily obtained we have used instead of salamanders live steam lines and we have found that to be more satisfactory than salamanders. Such a proposition is not possible where the plant is isolated from a heating line.

J. A. Turner.—How do you provide sufficient radiation?

Mr. Dunnells.—Simply by using a considerable number of pipes.

R. F. Egelhoff.—There is one point I think should be covered in this matter of heating of concrete, and that is the injurious effect to the finished floor by the gases from the salamanders. I would like to know how that is overcome, the softening of the top finish due to the action of the gases.

MR. UPSON.—It would seem to me that if the weep holes or the vents were large enough so the volume of air was sufficient to reduce the concentration of the gases that that danger would be obviated. I should think it would be only when the holes were so small they had a concentrated gas.

S. C. HOLLISTER.—In 1918 some construction work was carried on entirely enclosed, with steam escaping in the enclosure, and the temperature was maintained at 50 deg. for three days; the outside temperature, the mean temperature for the three days was 12 above zero. The presence of the steam not only maintained the necessary temperature, but also kept the air damp, which was advantageous to the curing of the cement. The concrete resulting—of course, the mixture was a rich mix—was equal to concrete of the same mix using the same aggregates and cement, under normal weather conditions.

W. F. TUBESING.—The biggest danger from salamanders is when you don't protect the bottom with sand, and the heat goes through and leaves a spot of probably three or four feet in diameter which is badly affected from the heat going down from the bottom of the salamander. We have to put about 3 in. of sand over the floor, and then set our salamanders on top of that.

MR. UPSON.—That is an important matter. How much sand do you use, Mr. Turner, for that?

J. A. TURNER.—Just two or three inches, for a radius of about five feet. That will prevent the ashes and heat from going down.

There is one matter I think it is well to mention further. That is the importance of heating the materials before the concrete is mixed. I think it is erroneously understood in some places that if you thoroughly heat your concrete after it has been put in place, you don't have to pay much attention to heating the materials before it is mixed. I remember a few years ago I had some tests made, at the University of Pennsylvania, taking aggregates from outside when the temperature was about 20 deg. and not heating them but mixing them up just as we found them, with cold water, and then putting them in inside, where the temperature was 70 deg., and I found that you couldn't get more than about 1000 to 1200 lb. strength on that concrete, even though it was kept from freezing after it had been mixed. Apparently there is some chemical action there that must take place when the cement is setting that can't take place if the material is chilled.

I also found at the same time that if you took the materials from the outside when the temperature was 20 deg. and heated the material and mixed your concrete and then instead of protecting your concrete, putting it outside and letting it stay in the cold, the concrete was stronger later than it was by the first method. I think it is very important that everybody understand that materials must be heated, the chill taken off, that is, if the heat is thoroughly kept up to 50 deg. at least before it is mixed. I think that is really more important than the protection of it afterwards.

STANTON WALKER.—In connection with heating the aggregates, I call attention to a method that was used on a job this winter with which I had some contact. Here they introduced live steam into the mixer; the aggregates were not heated, the water was heated. We had occasion to make some tests of this concrete, and were able to compare it with similar concrete made under normal temperature conditions. This was mass concrete, placed at temperatures below freezing, not extremely low temperatures, however. The only protection that was given the concrete was to cover the forms thoroughly with canvas, and very satisfactory temperatures were maintained inside the forms by this method. Apparently the temperature of the concrete when it went into the forms, together with any heat that may have been generated by the cement, was sufficient to maintain entirely satisfactory temperatures in the concrete. There were few frozen lumps.

Warm water alone is not sufficient to help the strength of the concrete. In some tests that have been carried out at our laboratory, in which we used mixing water at various temperatures, mixed with aggregates and cement at ordinary temperatures, we found very little effect, due to the temperature of the mixing water. In other words, the heat of the mixing water was very quickly dissipated in attempting to warm up the aggregates.

CONCRETING INCLINED SLAB.

What is the best way to concrete an inclined slab so as to secure the best results, both in regard to strength and looks, at the same time with facility in handling the concrete?

MISS KATE GLEASON.—I tried that and succeeded with burlap; put burlap over the bottom of the form, or the roof, if you are using it as I did, for a roof. I have had perfect success.

MR. UPSON.—On the inclined slab?

MISS GLEASON.—Yes. The burlap has enough holes to fix the wet concrete as you put it on; it goes on as fast as you can put it, it doesn't run at all.

MR. UPSON.—That is on the bottom of the slab or on the top of it?

MISS GLEASON.—On the bottom of the slab.

MR. UPSON.—Then you finish the top of the slab. The incline is not enough to make the material entirely run.

MISS GLEASON.—It doesn't run at all, it stays right there. I did it on the roof of one of my little houses. First I tried to put it on the top of the ordinary roofing material and it ran; so when I used the burlap without any underpinning, without any forms, and the burlap sagged somewhat, enough so that my material came too thick in places, it took more concrete than I wanted to put on; so the next one I did I put on a very cheap roof, made of odds and ends, the more nails the better, and put the burlap on that. Then I put chicken wire and simply poured the concrete on. When I sold the house I told the man that that was an ex-

periment, and if anything happened to his roof I would make it good, and it has gone through three frosts and thaws and doesn't show the least bit of trouble.

D. A. ABRAMS.—Up in the Iron Range where they ship great quantities of iron ore they use inclined bottoms for these ore pockets. The slope is so steep there that it is not practical to build them in place, so they build them separately and set them in place. That is one way to get around it.

MR. UPSON.—I have also had forms built on both sides and on the top of the form lay up one board at a time, paddle it in and lay up one board and paddle it in and lay up another board, on coke bins and steep inclines, where you have to have forms on both sides. That is slow and expensive work.

S. C. HOLLISTER.—During the construction of the concrete ships, in which under some phases of construction inclined slabs, and even curved slabs, were cast, the scheme was to cast the slabs with forms on both sides, the forms on the top side having a space of about 18 in. of surface up the slope, being kept in place, and the next 18 in. being a removable panel. That gave an opportunity to cast the concrete by introducing it into the forms, about every 3 ft. along the slope. In that fashion a good deal of the expense was reduced that would otherwise be present by building up the top forms as you went along. The percentage of steel was very high, the slabs were thin, the difficulty was great in casting a slab of that kind. Nevertheless, a consistency was used that would correspond to about a 2½-in. slump, and that is a very stiff mix. And an air hammer or a pneumatic hammer or an electric hammer was used, with a mushroom chuck to vibrate the bottom form, and that gave fluidity to the mass, and a very dense slab resulted. No trouble was found after the removal of the forms, either on the upper or lower face; the concrete thoroughly compacted about the steel and filled the entire space.

MACHINE TO MAKE SEPARATORS.

Would there be a demand for a machine, the cost of which would be very low, for turning out in quantity concrete separators and doughnuts for use in wall and column forms?

J. G. AHLERS.—We have a small machine we made ourselves, about four or five years ago, and it is still doing service. I don't believe it has cost us more than fifty dollars to make it. We made some brass molds and brass pins that go down through it; it is a very crude sort of a thing, but it does the work. These doughnuts we all make by hand, a little tin mold and form it in there and just set them aside to cure. I don't believe it would be commercially practical for anybody to go into the business of making these machines, because they would be so cheap and there wouldn't be enough of a market for them.

MR. UPSON.—Do you use them on the beams at all or on your slabs?

MR. AHLERS.—No; we usually use commercial metal separators that you can buy from a number of manufacturers.

R. F. EGELHOFF.—I have found that in using the doughnuts they are very apt to be broken in tieing them to the column bars and handling the column steel. We have changed to the use of a "U" separator, something which can just be dropped over the hoops on the columns and can also be used on the beams, dropping it over a ⅜-in. bar tied to the stirrups, thereby keeping the stirrups away from the beam side. It works very satisfactorily.

FOUR-CYLINDER GAS ENGINES.

Is the new four-cylinder type of gas engine more dependable than, and superior to, the old one-cylinder?

FRANK L. GINSBERG.—In certain machines the one-cylinder engine is still most successful. Up to 5 hp. we have found it to be universally the simplest and the easiest to operate. Above that we are getting into two- and four-cylinder engines, in our half-yard mixers and three-quarter yard mixers, and as high as 16 hp. and over, we are getting into the four-cylinder engine. Now various manufacturers have on the market large units, up to one-yard concrete mixers, which are being handled entirely by four-cylinder gasoline engines, displacing the old electric motor and the steam engine and paving equipment is now universally gasoline driven, by four-cylinder engines, of from 25 to 40 hp. We have even got them as far as steam shovels and big locomotive cranes that we laughed at two or three years ago. We are now using four-cylinder gasoline engines because of their simplicity and their economy of operation. We do away with licensed engineers and licensed firemen in some localities where that is rather expensive, night watchmen, and excessive overhead on days when you are not operating and still have to maintain power in these machines.

Getting back to the smaller units, I would say that the single cylinder engine in mixers up to one bag, say, is the more economical and the more easily handled unit than the multiple cylinder engine.

MR. UPSON.—In using gasoline engines for pile drivers we get away from a very important item, and that is the item of coaling a machine of that type. I was interested in observing the expense of operating pile drivers out in California. In New York it costs fifteen to twenty dollars a day to coal a pile driver. Out there it costs $1.25 a day to oil it. That is, the oil required to fire the steam engine.

REINFORCING DETAILS.

Is it not reasonable to ask and to expect that element of the architectural profession and some of the engineering profession who now furnish comparatively meager details of reinforcement, to show, in fully detailed and properly referenced drawings tying in details to floor plans, the arrangement of rods at points of intersection of beams and girders; also, notes with reference to steel in structural members such as 1½-in. square rod—50 per cent bent up at quarter points? Responsibility shirked by the architect is thrown upon the

contractor, who passes it along to the job superintendent, where it reposes as a misplaced burden. The extent of this weakness on the part of arcihtects may be judged by the action of the bar companies and their agents of setting up organizations to furnish purchasers with workable reinforcing details.

My observation of that is that a contractor now, in order to be successful, generally has to be able to supply those details. I would like to know what the rest of the audience thinks about it.

MAX DUNNING.—At the present time I think the great majority of the better architects are employing structural engineers to design their concrete, and there the reinforcing goes with it. There are, of course, some architects who hesitate to go to that expense, but they are the kind of architects that I think the contractor is also afraid to do business with. It has been my practice in this line of work always to make complete structural floor plans, to make cross sections of every beam on the job, to make typical elevations of beams that are different, to show the spacing of the stirrups; not necessarily complete bar details, but to give sufficient information to the contractor so that without any trouble and without any questions he can go ahead and make his complete job.

MR. UPSON.—I would like to ask you what your sentiment would be with reference to letting the work to two different concerns, one concern who had a complete engineering organization who could check your plans and co-operate with you and co-ordinate with you, or to the one who would take your instructions and follow them out without asking any questions? Which would you rather have?

MR. DUNNING.—I didn't mean that I didn't want the contractor to ask any questions, but I mean to give him sufficient information so that it really wouldn't be necessary. Now if he wants to ask them it is all right. I didn't mean that.

MR. UPSON.—I am in perfect agreement, there is no contention about it, but I was just bringing out the point whether the architect would rather have his medicine taken without asking any questions or whether he would rather have a man who would sit down with him and ask questions about it.

MR. DUNNING.—Any architect or any engineer would always welcome any constructive criticism.

L. H. ALLEN.—The question here seems to presuppose that an architect or an engineer shirks his job if he doesn't give a complete drawing given of every beam, and I suppose every bar in that beam. I think it is hardly fair to beg the question that way, as has been done here, and I don't think that the contractor has a right to expect such a complete drawing. On the other hand, I think the architect has a right to expect the contractor who undertakes a reinforced-concrete job knows how to place steel and how to assemble his reinforcing. When an architect details a door he doesn't show how the sash bars are mitred into the top rail, and all the other little details, because there is a supposition that the mill man knows how to make a door before he bids on the job. I think that any reinforced-

concrete contractor who bids on a job ought to know, and his superintendent ought to know, how to place his reinforcement before he undertakes the job at all. And it is economically impossible to give a complete detail of every beam on a job. It would seem to me that it is sufficient to indicate the reinforcement, and not show things such as the details of assembly, which are shown in any textbook, which any student should know.

J. P. Schwada.—I don't believe that the detailing of reinforced work can be compared to the detail of a door, because upon the details, the correct carrying out of the details of reinforced work, only by the successful carrying out of those details can we obtain a structure which is safe and installed as it was originally intended, while a little change in the door is really unimportant.

I have been giving this matter of detailing some consideration, and I have arrived at the conclusion that the engineer should be expected to show all the details. For two reasons. In the first place, it is the only way whereby he will obtain the job that he wants, and, in the second place, I am satisfied that in the long run he pays just as much for the work as if he were to detail it in the first place. On all important structures for the city of Milwaukee we show the complete elevation and plans, we show every bar, we number every bar, and then we make a bill of bars. In this bill of bars we show the number of the bar, and that number is shown on the plans. We also show the size, the length, the weight per foot, the total weight for the particular type of bars. We also show bending diagrams, and then we show just exactly where those bars go, and we find that we save considerable time not only in the office, but in the field. The contractor knows just exactly what is expected of him.

On the other hand, if you leave it to the reinforcing bar companies, they will charge you, as they do in Milwaukee, I think, about a dollar and a half a ton for making those details. Then those detailed drawings are brought back to the office and we are asked to check them, or, in other words, the responsibility comes right back to us. So we have all that pawing and mauling of work to be done over and over again. We find that if we do it in the first place the thing is carried all along, the bar company knows what we want, the contractor puts it in as we show it, and it assists the contractor and it assists our inspectors in the field. I think every engineer ought to show those details.

Quality on the Job.

The average construction man would like to have the American Concrete Institute determine the best method to enlighten the concrete foreman, who has in most cases not had the advantages of even a high school education, as to the use of water in concrete and also the grading of the aggregate. I have had occasion during the past year to take a number of our foremen to hear some of the lectures on the grading of aggregate and water content, and I am sorry to say

that practically all of these lectures were so technical that they were far over the head of even the more enlightened listener. I think that a real effort should be made to conduct a series of lectures in the different parts of the country explaining in simple language that the slump test is simply a means by which one can determine the amount of water that can be used, and also to inform them in simple language of the method of grading the aggregate to take the place of the 1:2:4 measured by volume which they have used in the past. I believe that an expression of opinions as to the best method to obtain these results would be well worth while.

. . .

The control of the quality of concrete is our greatest problem. Apparently your Committee C-6 intends to cover this subject. However valuable their report may be from an engineering point of view, it isn't going to help the job if it is in language that only an engineer can understand and appreciate. If you want better concrete on the job, get your reports out so a concrete labor foreman can understand and follow their recommendations. When the reports for this meeting are in, ask yourself the question if this is possible.

. . .

How much of a load can be safely placed on 1:2:4 concrete three days old?

STANTON WALKER.—The first two paragraphs of this question can be resolved into the following: "What can we say in simple language to the man in the field which will make him understand how to control the proportions of materials and thus obtain concrete of uniformly satisfactory quality, at the greatest economy?" I shall discuss the third paragraph under a separate heading.

In view of the several somewhat lengthy papers along these lines which will be read at later sessions during this Convention, I feel that a discussion of this subject this evening is somewhat premature.

I am further handicapped by not wishing to steal the thunder from a paper which Mr. Ahlers and myself have prepared, by discussing in too much detail what has been accomplished in job control. However, I have one or two thoughts which may be conducive to discussion.

For a simple statement of the problem of obtaining high quality concrete which everyone can understand, I cannot think of anything more terse and to the point than that made at our Convention in Cincinnati last year by Mr. MacMillan in which he said, in effect: "Mix the concrete with as little water as you can and as much stone as you can, and after it is placed keep it as wet as you can."

The application of this simple rule solves the problem of securing the proper slump and the most economical proportions of fine and coarse aggregate on the job. This rule, of course, needs some elaboration. The quantity of mixing water is reflected in the plasticity of the concrete and

will therefore be fixed by conditions on the job. The maximum proportion of stone will also be governed, to a considerable extent, by the nature of the work. Also, if the quality of the concrete is to be bettered, the coarse aggregate must not be added to replace cement; that is, the quantity of cement per cu. yd. of concrete should remain the same. I think it is possible that some of our friends have failed to recognize that this simple rule is the fundamental of practically everything that has been said and written about proportioning concrete during the past few years, and have allowed statements of the "why and wherefore" of this rule to cloud the main issue.

It is true that methods of calculating the proportions in advance of starting the work, which will fulfill this rule, may be somewhat more complicated than can readily be understood by the concrete foreman who has little or no schooling. On the other hand, we do not ask the reinforcing steel foreman, or the job superintendent to check our designs of reinforced-concrete members, so why should we expect him to check our designs of concrete mixtures?

As many of you know, I have been connected with a series of field investigations carried out in the vicinity of New York and Philadelphia during the past construction season. It is my conclusion as a result of these experiences that the better control of concrete lies not only in the education of the man in the field, but in the perfection of means of measuring quantities of materials.

When positive means of measurement are the rule, instead of the exception, it will be a comparatively simple matter for the man who is not capable of deciding upon the proper proportions to follow the instructions of someone who is.

A vast amount of valuable information on proportioning concrete has been distributed during the past few years. An earnest effort is being made by the Portland Cement Association, in particular, to give as wide distribution as possible to such information. They have recently adopted a policy of sending a number of field men from each of their district offices to our Laboratory for a short course of instruction in Laboratory methods and in the information obtained from our tests.

Men in the various engineering organizations are making serious studies of the problem of applying these results in the field. With all of these missionaries working in the cause of good concrete it cannot be long until the design of concrete mixtures is given the standing that it deserves; that of an engineering problem comparable with the design of the structure itself.

"How much of a load can safely be placed on 1:2:4 concrete 3 days old?" Obviously this question is one that does not permit a single answer which is applicable to all conditions. I have therefore based my answer on standard test specimens, cured and tested in accordance with standard methods, with the thought that from a comparison of their strengths at different ages, and a knowledge of the efficiency of concrete in structures

at the usual ages, reasonably accurate deductions can be made to fit each specific case.

Tests have been made at the Structural Materials Research Laboratory on concrete at ages of 1 day to 5 years for a wide range of mixtures and consistencies. These data are too numerous to report in this brief discussion. Many of them have been published in the Bulletins of the Laboratory and in technical papers prepared by members of the Laboratory staff.

In general, these tests have been made on 6 by 12-in. concrete cylinders stored in damp sand or a moist room at a temperature of about 70 deg. until test, and tested damp.

A study of the results brings out the interesting fact that the relation of the strength of concrete at one age to that at another is dependent on the strength of the concrete. This relation enables us to give an answer to our question which will apply to all classes of concrete. An average of a large number of our test results give the following percentages for strength at 3 days in terms of that at 28 days.

Strength of Concrete at 28 days lb. per sq. in.	Strength at 3 days Per Cent of Strength at 28 days
1000	20 to 25
2000	25 to 30
3000	30 to 35
4000	35 to 40
5000	40 to 45
6000	45 to 50

1:2:4 concrete of the consistency required for the ordinary building construction has a strength of about 2000 lb. per sq. in. when tested in 6 x 12-in. cylinders and cured in damp sand until test. At 3 days we might therefore expect a strength of about ¼ to 1/3, or 500 to 600 lb. per sq. in. for normal temperature conditions.

It is important to know, however, that the early strengths are greatly affected by the temperature at which the concrete is cured.

For an investigation in which concrete specimens in sealed cylinder forms were placed in water at various temperatures immediately after they were made, it was found that at 3 days the strength of the concrete cured at 65 deg. F. was about 5 times that cured in ice-water (33 deg. F.). The 28-day results were affected to a much less degree. Following are the strengths obtained at 3 and 28 days temperatures ranging from 33 deg. to 212 deg. F. expressed as percentages of the strengths at 65 deg.

Temperature of Curing Water Degrees Fahrenheit	Per Cent of Strength of Concrete Cured at 65 deg. 3d.	7d.
33	22	72
42	36	78
65	100	100
125	177	108
212	207	102

The problem of obtaining high early strengths has interested engineers for a long time past. As a result considerable thought and effort have been expended toward developing a means of accelerating the hardening of portland cement and in the production of special cements which will attain their strengths quickly. During the past few years the market has been flooded with special compounds, under one name or another, to be mixed with portland cement concrete for which the general claim is made that the strength of concrete will be increased, particularly at the early ages. Probably the best known of the materials is calcium chloride, and many of the proprietary compounds consist chiefly of this material.

An exhaustive study of the effect of calcium chloride has been carried out by our laboratory. These tests show that for the ordinary mixtures the strength of concrete at 2 days may be increased as much as 50 per cent. This beneficial effect is much less evident at the later ages. At 28 days the maximum increase in strength was approximately 10 per cent for ordinary mixtures, and still less at the later ages. The best results were obtained for 2 to 4 per cent of calcium chloride in terms of weight of the cement. At the same time a study was made of a number of the proprietary compounds with calcium chloride base. Essentially the same results were obtained. It is interesting that in general their effectiveness could be measured by the quality of calcium chloride which they contained.

Considerable progress has recently been made in France in the development of high alumina cements which attain practically their maximum strength at 3 to 7 days; and this strength is almost double that of portland cement in concrete of similar proportions at 28 days. Col. Spackman, I believe, will discuss this material in detail on Thursday morning.

In Italy a method of making concrete for roads from very dry and rich admixtures has been advocated and, I understand, used with considerable success. The process of manufacturing the concrete is patented, and consists of ramming into place a very dry mixture consisting of a special cement and 1¾ parts of crushed granite ranging in size from about ½ to 1½ in. As would be expected very high strengths are obtained at the early ages. Tests made in our Laboratory using local aggregate and cements, mixed according to the formula, gave higher strengths than the Italian materials. It should be stated, however, that the Italian cement was about 1 year old when the tests were made. It seems that this method of making concrete has considerable possibilities in repairing pavements, and in certain other special instances.

W. P. BLOECHER.—It seems worth while to call attention to the fact that on paper it is very easy to demonstrate that weighing your aggregates ought to give you a good deal more accurate results than any volumetric measurements. The volumetric measurements, of course, are incorrect by reason of the bulking effect, particularly in the sand, and we have lately found out that they amount to a whole lot, whereas weighing will sort of automatically smooth those out, because the effect of the weight of the water is reduced to about one or two per cent, whereas the effect of the volumetric correction on account of the presence of water is a much greater percentage than you get on a basis of weight.

R. W. WEITZ.—We use a piece of equipment for proportioning that I think is probably a little bit different than usually seen in this country, because we imported it. It consists of two cylinders, it can consist of three, one in which we put the cement, one the sand, and one the rock or pebbles. The bottom part of the cylinder revolves. There is an opening in the bottom plate that lets out the exact amount of sand or gravel or cement, in each cylinder they come out simultaneously and fall into a continuous mixer, one of these screw type, and the proportion is always exact. The mixer, of course, being a continuous mixer, we have to let the batch wait in the hopper until the hoist bucket can go to the top of the tower and dump and come back and load again. Our water is added by a sprinkler. That, of course, can be adjusted so it is also steady, and we get an absolute mix all the time.

FORMS.

It would seem to the writer that we will eventually have either to standardize our designs to a greater extent than has been done in the past to permit the re-use of forms, or work out some method which would involve the use of more metal instead of lumber. This question, I believe, is far more important than plant layout, and I think it is worth serious consideration by all those interested in reinforced-concrete work.

Can and should the present type of wood forms be improved so as to eliminate the necessity of pointing up after the forms have been removed?

L. H. USILTON.—The question of forms, from a money standpoint, and labor, is the most important on the ordinary concrete job. The question here raised is the matter of design, so as to avoid as much of the formwork as possible. That can be done very often by sacrificing a small amount of steel or concrete. It can also be done where we cannot avoid changing the sections by planning the formwork from the beginning of the job, making up the sections in such shape that they can be easily altered. The difficulty lies not alone in the actual cost of the formwork, but in the savings in the amount of lumber used, often in the saving in time on the job, because of the fact that when the forms are planned to use over

and over again you can maintain a certain speed per floor, with the same regular gang, without any interruption. A day lost often means a week, particularly if you do not finish a floor on Saturday or on the previous day. It also leads to unbalancing of the job in that more floor men have to be put on in case of some special formwork or else that the formwork drops behind so that you have an excess of laborers on the concrete and on the steel.

The item of pointing referred to in the second paragraph under Forms could not be entirely eliminated by any improvement in Forms. There is a certain amount of pointing due to improper spading of concrete which could not be eliminated at all.

The question whether wood forms can be improved so as to cut out any pointing beyond this depends on the character of building desired. I think that with the ordinary formwork where they are cut in fairly good shape and where they are well oiled each time before using that a sufficiently good finish can be gotten for the money actually to be spent. If a finer finish than this is desired, I think it is much cheaper to plaster it than to do anything else.

C. E. LOCKE.—I would like to give one instance of a ten-story building where there were four different floor loads required, and in order to design that building for the best use of forms the contractor kept the width of beams and girders the same from the first floor to the roof. The lower floors had beams and girders of a greater depth. When he came to the first floor, which had only about a 150-lb. load, they cut the depth only of beams and girders, and took the slab to a floor above, kept the same forms exactly, but added 1 in. on the top of the slab and increased the steel. Then in the floors above they decreased the steel again to the 150-lb. load and in the roof decreased the depth of beams and girders and slabs, but not the roof, and changed the steel to suit the roof load, thus eliminating many changes in forms.

MR. UPSON.—I wondered if anyone here has been using the beveled edge forms. I have seen that used from time to time. I wonder what the experience is.

A. W. RANSOME.—We always used it, for a great many years. The matched boards were cut back on a 10-deg. bevel on both edges. That brought the points together and we have no trouble with the material getting into the joints. The swelling of the lumber closed them up.

MR. UPSON.—Is there anything more on this?

MR. RANSOME.—One other point that I might bring out in that method of construction. It is very much easier to put up your temporary shores when you are planning on removing the main shoring under your floor slab. You can put a few shores directly on one row of 2 x 6 of the B. & M. material that you are using for the shoring, and if you drive that up reasonably tight, that will keep the one row of 2 x 6 up, and the rest of the sheathing and construction you can drop very easily.

J. A. GARROD.—It would not seem to us that designs for reinforced industrial concrete buildings can be standardized at this time so as to allow of the re-use of forms from job to job. The variety and diversity of requirements in these buildings seem to preclude any considerable standardization. The mill or factory operator looks upon a building as a tool that should absolutely suit his requirements—not a tool that may be partly right and which will tend to increase the cost of the work produced in it during its entire life. The standardization should be carried to the limit in the various units and floors of any project under consideration. It is usually economical we believe to use up the forms on the job where they are made.

Our experience shows that an inclined slab can be satisfactorily concreted with an angle of 30 deg. to the horizontal by making a stiff mix and carefully placing without any forms on the top surface. At a greater angle to the horizontal up to at least 45 deg. it is satisfactory to apply light boards with comparatively few braces and liners to the upper side of the inclined slab as the concrete is placed.

At angles of 60 deg. with horizontal and greater angles it has proved in our experience to be most practical to form both sides of the concrete slab in the same way that a plumb wall is formed and braced.

Our usual methods of operating is on the American or open shop principle. Working under this principle we have found it helpful and productive of good results to offer extra pay for extra effort on the part of the workmen. The man is assured of his regular hourly pay as remuneration and some definite and understandable scheme is set up which can be readily applied to the work in hand, by which he is enabled to make 20 per cent to 30 per cent more money in the same time in payment for extra effort put forth.

We understand from union operators and from by-laws and working agreements of unions which we have examined that the unions and unionized labor does not favor and will not permit its members to work on any bonus system which involves measuring work and paying for extra efforts put forth for extra remuneration.

We believe that forms as at present constructed are made so tight and regular that by chipping off fins and irregularities, going over the piece of concrete immediately affected with suitable air-chipping hammers, there will be no necessity for pointing, providing the architect or owner is willing to accept this kind of finish rather than the pointing patches. The tooled surfaces have a different texture to the work applied with a trowel, but after painting, as is usually done on the interior of the building, they do not present difference and have the decided advantage that they will never peel or become loose as patches are very apt to do.

Such treatment can also be used on exterior walls where it is not objected to, but the usual method as recommended by the Committee on Treatment of Concrete Surfaces applied to the exterior of the building gives a good general effect for the exterior treatment of buildings.

FLOOR FINISH.

Assuming that only good ingredients be used for finish, what procedure in the placing of 1-in. finished floors afterward on the rough concrete slab will insure finished floors which will not crack and loosen from the rough slab? To what extent does the proportion of the mix for finish affect the loosening up of the finish from the rough slab due to difference in cofficients of expansion in the rough concrete slab and in the finish slab?

Has anyone ever laid a concrete floor satisfactorily without pouring it monolithic?

. . .

Assuming that only good ingredients be used in mixing finish, is it possible so to formulate the method and practice in placing monolithic finished floors that the construction superintendent can in all cases be absolutely sure that the result will be a first-class durable floor? The solution of this problem should take into account the following varying conditions:

a. Varying weather conditions.

b. The need of putting in large areas of finished floor in a single day.

c. The factor of speed in construction which makes it necessary that workmen get on the finished floor within two days after finish is placed to erect forms for the next story.

d. Placing finish in winter when it is necessary that heat be applied to the slab both bottom and top. How can dusting of the floor or a soft finished surface due to the action of the heat and the gases from open fires be avoided?

Is there a cheap dust that can be put on top of finished slabs so that the droppings from the slabs or pour above will not adhere to finished slabs underneath, as I have found sawdust, mill shavings or sand do not properly take care of these droppings?

Why do not engineers and architects more generally advocate the placing of a 1-in. floor finish at a later date, rather than monolithic with the structural concrete?

E. D. BOYER.—I would like to hear someone answer this question: Has anyone ever laid a concrete floor satisfactorily without pouring it monolithically?

L. F. FAIRCHILD.—The Eastman Kodak Co. has about half a million square feet of floor space not laid monolithically, and it is very satisfactory. Our specifications state that a floor finish laid on the ground floor slab may be laid monolithically, but in all other cases it must not be laid monolithically.

MR. UPSON.—How is it done?

MR. FAIRCHILD.—Laid after the structural slab has hardened.

MR. BOYER.—How is the surface finished?

MR. FAIRCHILD.—The structural slab is roughened, that is, the structural slab is designed to carry the load independent of the finish, and is roughened at the time or is picked afterwards.

MR. UPSON.—Then how thick is the finish?

MR. FAIRCHILD.—One inch.

MR. UPSON.—How do you proportion it?

MR. FAIRCHILD.—It is a 1:3, I think.

MR. UPSON.—Do you use any hardener in it?

MR. FAIRCHILD.—It is a patent finish in a good many cases.

MR. UPSON.—How soon after the first part of the floor is laid is the top finish applied, and what is the preparation of it before the top finish was applied?

MR. FAIRCHILD.—The structural slab is roughened at the time that it is poured, and if it is not sufficiently rough at the time they wish to apply the finish, why it is picked in order to roughen it.

MR. UPSON.—Then you must use a patented floor, do you?

MR. FAIRCHILD.—We have had better experience with the patented floors than we have with the floors that we have put down ourselves, although a great deal of the slab has been placed by ourselves and has shown fair wearing conditions, wearing surfaces, but in recent years we have been using the patented slab, patented finish, I would say.

MR. UPSON.—Do you use heavy trucks on that floor?

MR. FAIRCHILD.—No, I wouldn't say heavy trucks, although there is a great deal of trucking, trucking a weight of probably 500 to 600 lb.

MR. UPSON.—Rubber-tired vehicles?

MR. FAIRCHILD.—No, sir; some rubber-tired and some metal-tired.

MR. UPSON.—I would like to ask, Mr. Turner, what your experience is in that.

J. A. TURNER.—We lay floors both ways, monolithic and lay them afterwards with the 1-in. finish. Most of the work in the past I believe has been done more with the monolithic finish than with the finish laid afterwards, particularly because of the fact that you get a perfect bond and you don't have to worry about the slab coming loose; you do get the floors scarred sometimes quite a little by it, but if the surface is good, a few scratches that the floor gets when it is green hasn't particularly hurt it except for looks. But recently we have found that by exercising proper care we can put a 1-in. finish on later which bonds properly, it doesn't come loose, and gives equally good service, without the marked effect from the scratches and scars. Of course that is much better, too, when the cold weather comes on, because you can lay your finish then after the building is closed, so we have had equally good success both ways.

MR. UPSON.—What mix do you use for that?

MR. TURNER.—We have been using recently a mixture of about 1 cement to ¾ sand, to about 1¼ in. of clean grit.

MR. UPSON.—About 1:2?

MR. TURNER.—Yes, about 1:2, with more of the grit than sand.

MR. UPSON.—What provision do you make for expansion on that? The same provision as you made in the building structure?

MR. TURNER.—We don't make any provision, except where we put the finish down afterwards, we attempt to make the joint over the construction going below.

S. C. HOLLISTER.—It seems fitting to give some definite rules for the bonding of a 1-in. finish to the floor beneath.

In the first place, the concrete slab underneath shall be rough, should be clean of all dirt or dust or any loose particles of any kind, or chips or shavings or sawdust or any other material of that nature. It should be free from laitance, that is, the white, chalky substance that comes to the surface of the floor slab when a considerable amount of water, particularly, is used in pouring the floor. When the floor is properly cleaned and presents a rough surface, the next thing to do is to see that floor has taken up all the water it will take up. That is done by the sprinkling of the floor at intervals until the floor has apparently absorbed all the water it can, until it is saturated. That will take 24 to 36 hours of treatment of that kind. Then when you put down the top it is not necessary, although some seem to think it is, it is not necessary to put down dry cement on that surface—better not, better to put down the topping, the mix that Mr. Turner suggested is excellent, in general a one-to-two mix, one of cement and two of aggregate, and the aggregate may be divided up into two different classes, as Mr. Turner recommended. That should be troweled on carefully, and with whatever smooth finish is desired to finish it off. A floor of that kind, put down in that way, will develop a satisfactory bond.

Care should be taken that at the time of placing the topping there is no excess water standing on the slab. Any excess water will cause trouble at that point.

TIME OF MIXING.

> *The question is as follows: "I would like to know what strength is added to concrete by mixing the batch 1½ to 2 minutes as against 1-minute mixing, using a concrete mixer 16 to 20 r. p. m."*

D. A. ABRAMS.—The Structural Materials Research Laboratory, Lewis Institute, Chicago, carried out a series of tests a few years ago which furnish the answer to this question. The writer presented a report on these tests at the 1918 Convention of the Institute under the title, "Effect of Time of Mixing on the Strength of Concrete." A double-cone batch mixer of 3½ cubic feet capacity was used, with mixing times ranging from 15 seconds to 10 minutes.

If we plat the strength of concrete as ordinates and the time of mixing as abscissae, we secure a series of curves which may be represented by the equation—

$$S = k + n \log t$$

Where S = comprehensive strength of the concrete.

t = time of mixing (seconds).

k and n are constants whose values depend on the mix, consistency, age of concrete, and other conditions of tests.

For compression tests of 1:4 concrete, in the form of 6 x 12 in. cylinders, relative consistency 1.10, using pebble aggregates graded 0-1½ in., and tested at 28 days, $k = 900$ pounds per sq. in. and $n = 750$.

Under the conditions of these tests we obtained from the above equation the following strengths for concrete mixed ½, 1, 1½, 2, and 4 minutes:

Time of Mixing Min.	Sec.	Compressive Strength of Concrete at 28 days—lb. per sq. in.
½	30	$(900 + 750 \times 1.477) = 2000$
1	60	$(900 + 750 \times 1.778) = 2230$
1½	90	$(900 + 750 \times 1.954) = 2360$
2	120	$(900 + 750 \times 2.079) = 2460$
4	240	$(900 + 750 \times 2.380) = 2690$

Other conditions being equal, mixing for 1½ min. gave an increase in strength of 130 lb. per sq. in. over the 1-minute mixing; or an increase of about 6 per cent. Mixing for 2 min. gave an increase of 230 lb. per sq. in. over the 1 min. mixing; or an increase of 10 per cent. Results of a similar type were found in tests on other ages, mixes, etc.

For usual mixtures, consistencies, etc., doubling the time of mixing increases the compressive strength of concrete about 10 per cent, regardless of the mixing time considered. This makes it doubtful whether we are justified in requiring more than 1 min. mixing (after all materials are in the mixer) on work in which the output of the mixer dictates the quantity of concrete placed.

The effect of time of mixing was not uniform over a wide range of proportions. It was more effective for the lean mixes than for the richer; more effective for the dryer mixes than for wet; more effective for the small sizes of aggregate than for usual concrete size. The increase in compressive strength of concrete at 28 days due to *doubling the time of mixing* showed the following ranges:

(1) 14 per cent for 1:9 mix to 8 per cent for 1:2 mix;
(2) 15 per cent for relative consistency 0.90 (1:4 mix) to 5 per cent
(3) 30 per cent for aggregate 0-14 sieve (1: 4 mix) to 8 per cent for
for r. c. 2.00;
0.2 in.

J. A. TURNER.—Have you any data on a less time of mixing?

MR. ABRAMS.—These tests were turned out for mixing times ranging from 15 sec. up to 10 min., so we have complete data throughout that range. The same relation holds throughout. If you should mix your concrete 30 sec. as compared with 15, you will get 10 per cent increase over 15 sec. mixing. It is not practical to attempt to mix concrete less than 15 sec., because the time required to get the batch out of the mixer was a large proportion of the total time.

Products Plant Problems.

Discussion of Products Plant Manufacturing Problems by and for Concrete Products Manufacturers.

A. J. R. Curtis in the Chair.

Mr. Curtis.—The concrete products industry is going through a transition from old and slip-shod methods to new and scientific methods of manufacture. Small scale manufacturing operations of yesterday are giving way to larger scale operations to meet the tremendous demands of the present and of tomorrow. The crudities which marked earlier experience with concrete products are changing to beautiful and refined expressions in a way almost as miraculous as that by which the ugly worm becomes a beautiful butterfly.

This session is called particularly for the discussion of problems of manufacturing economy. The strictest economy in manufacturing processes is an absolute essential at the present time. No manufacturer is safe for a day or even an hour who is not practicing the greatest economy in the manufacture of his products. Present day competitive conditions demand that we give this phase of our work more thought than ever before, nor are we sure that competitive conditions in the future will not be much more strenuous than they have been up to this time. It is the belief of many that we are just on the edge of competition now. We have dabbled in the game enough to really attract the attention of our competitors, and from now on it behooves us to study well manufacturing efficiency.

At this point J. W. Lowell, chairman Committee P-6, on Concrete Products Plant Operation, presented the report on Economical Manufacturing Processes printed on p. 633 of this volume.

E. W. Dienhart.—We are fortunate today in having methods available whereby we can quite definitely measure the efficiency of an aggregate as regards its concrete-making properties, particularly with regard to its properties for making high strength concrete. The committee's report showed that the fineness modulus of an aggregate—fineness modulus simply means the measure of the fineness of the aggregate—can be determined by screening material through a given set of sieves.

Fig. 3 of the committee's report gives the results of a comprehensive series of tests made on concrete brick. It will be noted that the grading of the aggregate had a marked influence on the strength of the brick for a given amount of cement used. Within the limits of these tests the strength increased materially with each increase in fineness modulus.

In concrete products manufacture we are working with a very small range of grading of aggregates. We are limited in the maximum size of the aggregate by the thickness of the thinnest section of the product which

we are making, and for concrete products three-quarters of an inch as a general thing is about the greatest maximum size of particle which we can use. Accordingly, we must work with comparatively low values of fineness moduli. For instance, our maximum fineness modulus in this series was 4.75. This was an aggregate so coarse that it produced a very rough looking product, a product which would hardly be salable except for special cases. Were we to use larger sized aggregate, say up to inch and a half, which is commonly used in building roads and bridges of reinforced-concrete, we could have used much higher values of fineness moduli, using up to 6.00 and 7.00. The high value used in making these brick is about the lowest usually considered in a discussion of this same subject applied to the other types of construction. While the limits of concrete products manufacture prohibit the use of those higher values of fineness modulus there are many producers who are working considerably below the possible limits, for instance, many manufacturers of concrete building tile, block and brick use straight sand for aggregate. Those manufacturers are not obtaining 50% of the maximum strength of concrete that could be obtained by adding coarse aggregate to the mix. This is indicated by the curves.

A MEMBER.—Suppose that the coarse aggregate costs them more than they saved?

MR. DIENHART.—Naturally if the coarse aggregate is very expensive it might not be advisable to use it in place of the additional amount of cement required to get the strength necessary, but the use of these data will give you the information that you need to determine which is most economical. It will be readily seen that, for instance, in making concrete brick, meeting the standard requirements of the American Concrete Institute, 1500 lb. per sq. in., that a brick of that quality could be made with a 1:5 mix, using aggregate graded 2.75 fineness modulus. If you can use a coarser graded aggregate, say 4.25 fineness modulus, you can use a 1:9 mix. It would be quite an expensive coarse aggregate which would offset that saving in cement.

E. W. HILKER.—Is it ever justifiable for a manufacturer to decrease the quality of his product?

MR. DIENHART.—That depends on whether the manufacturer is making a product that is too good or not good enough to meet certain definite conditions. These conditions determine what the standard product should be. When the standard is set, there is no economy or justification in making a product that far exceeds the standard. There is nothing in the products business that will justify the reduction of a standard quality.

PHILIP WEISS.—I would like to ask if anybody has had any experience with the use of hydrated lime?

JOHN W. LOWELL.—Any material that is finely powdered and inactive, added to an aggregate that already has enough of fine material, is going to decrease the strength, because it will increase the amount of material passing through the 100 mesh sieve. If the aggregate recommended in this specification containing not more than 10% through the 100 mesh

is used, you will not need to use any other material, whereas, if you have an aggregate that has not enough of finely powdered material you may aid it by adding stone dust, or any other finely divided inorganic material.

W. H. CAREY.—Personally I do not believe it is necessary to use admixtures if you have a well-graded aggregate. To meet the objection of the gentleman who questioned the lowering of the quality, I think he was under a misapprehension. There was no idea, I am sure, on the part of Mr. Lowell or Mr. Dienhart, to suggest the lowering of the quality; it was simply a more economical way of maintaining or increasing the quality. It is a scientific proposition. By the way, gentlemen, I wonder how many of you realize that it is only recently, within five years, that the concrete industry has had the advantage of scientific research, since Prof. Abrams' first report was made in 1919? That is the first real help that we have had in the concrete industry to guide us in the manufacture of concrete products. There is no excuse now for any manufacturer to go it blindly; the full information is accessible free of cost, through the work, the combined efforts of the Portland Cement Association, which is putting up the financial end of it, and the Lewis Institute, under Prof. Abrams and the Research Laboratory, which is conducting the actual research work, to determine just exactly what will bring a good quality of concrete in the most economical way. Any man in this industry can have all of that information. It is free. All you have to do is to write to your nearest Portland Cement office, for instance, or to the home office at Chicago and they will gladly furnish it. The thing, as Mr. Lowell said, is that we who are engaged in this industry must know what we are doing, must know how to make products economically, how to take advantage of every feature, not be subject to an excessive cost and subject to guess work. We should know what we are doing, and there is no excuse for a man in the business now not knowing what he is doing.

A. J. R. CURTIS.—It seems to me that Mr. Carey has hit the nail on the head as to what this report really means. Today we are having very definitely specified for us the strengths which our units must sustain; the strength is given as 700 lb. We know that is the strength we ought to meet; it is absurd for us to have our units go to 3000 lb., if the strength required is 700 lb. We would not build a steel bridge to stand a load of 3000 lb. if the load required by the specifications was 700 lb. The whole substance of this report is finding a gradation of aggregate which will be the most economical that we can use for the purpose in view; and having found it, then the required strength is obtained by using a definite proportion of cement.

H. A. DAVIS.—The general principles set forth by this report are exceedingly valuable, if they are followed with judgment. It is important that the products manufacturer study the report carefully, so that the principles will be applied intelligently. For example, in the laying of concrete brick suction is necessary. This, therefore, controls the extent to which grading of aggregates in making brick may be applied.

MR. HILKER.—I would like to ask Mr. Lowell a question on the wet

and dry mix. In your judgment, would you obtain a better result by three minutes absolutely dry and one minute wet, as against one minute dry and three minutes wet?

MR. LOWELL.—Mr. Hilker has brought up an interesting point on which there is very little authoritative data. The committee, however, hopes to make tests the coming year on this problem. In the meantime it has recommended that the concrete be mixed two minutes after all materials and the water are in the mixer. It would be desirable to allow a total of five minutes per batch in figuring the output per day.

MR. HILKER.—We have conducted quite a few tests in our plant, mixing it perfectly dry one minute and then putting the moisture in, getting the water mark and flakey substance out and mixing it wet for three minutes; then we have mixed it dry two minutes and wet one minute, etc., and at each mixing we would discharge the mixer, two or three turns of the paddle. The three minutes dry and one minute wet, the maximum compressive strength in 28 days was 1055 lb. to the square inch as against the one minute dry and three minutes wet, 540 lb.

M. W. LOVING.—It is the practice in a number of the most modern plants to dry mix the material for three minutes, and then after they introduce water in it, to mix it an additional three minutes. By providing sufficient mixing equipment they have been able to improve and get a very uniform high-grade concrete by that method. When I speak of high-grade concrete in machine-made sewer or culvert pipe, that means 6000 lb. compressive strength to the square inch; so that the method of mixing dry to get the materials thoroughly mixed and then mixing for two or three minutes after the water has been introduced certainly gives an excellent and uniform high grade of concrete, and that is what the modern plants are doing today.

MR. HILKER.—Do you put any lime or other foreign substance in with your cement?

MR. LOVING.—The only place I have seen that done was in California in San Joaquin Valley in an irrigation pipe plant. This manufacturer uses pulverized limestone, claiming to get workability, and he claims to get stronger and more dense concrete by the process. There are at least a dozen other manufacturers producing pipe of the same quality who are getting this same grade with either granite dust or portland cement. It is just a question of grading your aggregate, as shown by the committee's report.

MR. HILKER.—Limestone and granite would be the same as aggregates, but in hydrated lime you are getting into a foreign material.

M. W. LOVING.—Where hydrated lime is used, it is to increase the workability. Some of the plants may be using it to some minor extent for that purpose.

JACOB BOSCH.—Although digressing somewhat from the report I should like to seek information regarding attractive facings for concrete units. Architects with whom I come in contact seem to think that we should manufacture block which lay up to make inexpensive attractive

walls, comparable in cost with concrete block stuccoed. We have down face machines in our plant and frankly we have not been able to produce what the architects want. Perhaps some discussion of this problem may develop information of value or interest in future investigation.

MR. LOWELL.—At the recent meeting of the Wisconsin Concrete Products Association at Milwaukee an architect representing the Architects Small House Service Bureau urged the concrete products manufacturers to develop inexpensive attractive block facings. He was quite interested and had been working with some manufacturer in producing such a face, not to resemble natural stone, but to have a pleasing surface worked out by texture treatment and shade coloring. A house was to be built with these new block. Attractive clay brick wall surfaces are produced through study and investigation by the clay brick manufacturers individually as well as through their associations. Generally speaking, I believe it is true that very little of this kind of study has been made by the concrete block industry. We collectively and as individual manufacturers should encourage architects to co-operate with us in solving this problem.

A MEMBER.—In answer to the question of Mr. Bosch: There is a manufacturer in Peoria making satisfactory faced block on a down face machine. Here is exactly the way he does it: he takes one part of cement to three parts of No. 3½ Crown Point spar and mixes it quite well for a facing material, putting that in first and tamping the backing in; he tamps a little more on the face block, than the ordinary block. As soon as it is taken out of the machine, he takes a fine spray and sprays over that. The number 3½ spar is so coarse that when the water washes the cement down, it does not leave the face streaked and the water soaks into the face. Those block come out absolutely straight edged and square cornered. Ordinarily we find it impossible to spray a block laid with the face on the side, because it will run, but if you are using a number 3½ spar there is no fine material and all you are doing is washing the cement back into the block, and you have an absolutely dense surface, but no run on the face of the block. The facing is about three-eighths of an inch in depth.

R. F. HAVLIK.—It is a good many years since I made any block face down. I am making them by another process at the present time, but I have always contended that you could make good block with any machine if you put the right kind of effort into it.

A. J. R. CURTIS.—I do not know how much crazing affects most of you. If any of you make architectural stone, you will run into it more or less. I have faith that we will find all the elements we will have to find to entirely eliminate crazing, because we do so frequently find examples of concrete that are not crazed. There are many methods to control crazing proposed by different people, and each one thinks he has the problem whipped, but when applied to someone else's practice, the same treatments do not always work, and so it has been an elusive thing. I would like Mr. Havlik to open the discussion on this subject. Mr. Havlik read a fine paper on this subject last year, and there are many here now

who did not hear what he had to say and do not know the method he used to overcome crazing.

MR. HAVLIK.—We have succeeded to a certain extent with certain textures. To make this clear, I will state just what aggregate we use. We face all our products with crushed granite and white portland cement. We use equal parts of 3, 3½ and 4 spar. The block are made usually face up, but the same thing would hold true if they are made face down. I do not know that the method has anything particular to do with the results. We also make trim stone by tamping processes and wash all our products with acid or dip the cast stone in acid. That, I think, would answer Mr. Bosch's question as to how to get a desirable surface and get stone with sharp edges. He tells me he has had difficulty in making the corners stand up. Let the products get hard, hard enough to handle satisfactorily by whatever method you wish to use for surfacing. I know if you let them get real hard and dip them in acid, you will get results. Some of you may say that is too costly, but it is not. We are doing a $35,000 job at the present time in concrete stone, which we figure to make by the tamped method. We found that much of it had to be cast, so the result is that probably over two-thirds of it is being cast wet, the balance tamped. We take both kinds of stone and wash it in acid, in the case of the tamped stone, and dip it in the case of the poured stone; and we have discovered that our cost is less than we based our estimate on when we bid on the job. I am a firm believer in eliminating all fine aggregates from facings. I will not go into the technical side of this; I probably do not know enough about it, that is my principal reason, but we have found out by a process of elimination, that certain sizes of aggregates do enable us to get stone that does not check. By the way, on this $35,000 job we are under bonds for two years to replace any of the stone that checks, and I assure you gentlemen that I would not furnish that bond unless I was positive that I knew what I was about in making that stone with that particular material. I think the same results can be obtained with other aggregates, providing you eliminate the very fine. Now, to emphasize that point. There are some cities in the East where two or three concerns are making beautiful high-grade concrete stone and they make it by the poured process and they machine tool it, but now and then you will find a piece checked, and I believe the difficulty is due to the thing I mentioned a while ago, that they use too much of that fine material. I always examine concrete stone to find the checks, because those are the things the architect objects to and you would object to if you specify that material for a job, and the architect is putting his reputation on your product when he recommends it for a building. Architects would use considerably more cast stone, if they were assured it would not check. The manufacturer who will guarantee his product with a bond will be favored.

I forgot to mention the cement content in our facing. Aside from the aggregate used in facing I find that we must use as little cement as possible. I do not mean 1:10 or something like that, but not 1:2. Formerly we used a 1:2 mixture, and occasionally both the cast stone and tamped

stone would check, but now we use a 1:3 mixture. That is rather lean for a facing, as most of the products manufacturers here will realize, but that gives us watertight concrete. We also manufacture garden furniture and it holds water, yet we use no waterproofing whatever.

A. J. R. CURTIS.—A subject which several people have asked about of late is, what causes efflorescence on block and other products, and how can it be prevented or removed? That is a subject on which there ought to be some discussion.

G. W. PORTER.—We had a little experience with that this past year. In Milwaukee there is a firm making a large quantity of concrete brick. They had noticed that efflorescence at times appeared on their brick. Barium carbonate was suggested as a preventive for the efflorescence. From 1 to 4 per cent. by weight of the cement was used. About 30 brick of each type and color, using the same grade of coloring material that had been used were made. About the time the tests were started the plant began using a better sand. This sand was also used in making the test brick. All of these brick were placed outside, some laid up in lime mortar and the rest without mortar. They have been examined frequently for the past six months. As yet, no efflorescence has appeared. I think, however, that this is due in large part to the better grade of sand that was used.

MR. DIENHART.—With regard to barium carbonate, we have looked into that problem in some detail, and while it may be true that in clay products barium carbonate will retard efflorescence, the chemical action that takes place when the clay products are burned is given as the principal reason for preventing efflorescence. Now our chemists say that the process or the chemical action of barium carbonate on cement is not at all similar and can hardly be expected to give those results. I just bring that out as a point of information on the subject.

A. J. R. CURTIS.—The next subject pertains to the application of stucco to concrete block surfaces, which to me is one of the most important subjects that has come up.

F. E. GARDNER.—In view of the fact that the Institute specifications on stucco covers the application of stucco thoroughly, I do not think it necessary to discuss application itself. I feel, however, that emphasis should be placed on the various stucco finishes that have been developed. Architects are studying the possibilities of unusual stucco finishes and very attractive results have been obtained. It would be difficult to describe all of these finishes, but as they are clearly shown in the Portland Cement Association's new booklet on stucco, I would recommend a careful study of the booklet, particularly as it is a reprint of the Institute specifications on stucco.

MR. CURTIS.—I would like to hear a few words from Mr. Lockhart. He had charge of the construction of the Home Sweet Home house at Washington, which was built in six days.

W. F. LOCKHARDT.—I think in general our best stucco jobs are those that follow most nearly American Concrete Institute recommended prac-

tice, which is based on a long series of tests covering a period of eight or ten years at the Bureau of Standards in Washington. The problem is one which seems comparatively simple. It comes down to two known facts: first, that rich mixes shrink more than lean mixes, and that wet mixes shrink more than dry mixes. The American Concrete Institute recommended practice is, of course, the usual 1:3 mix; the question of wetness is a harder matter to control. The mechanic naturally wants the stucco fairly wet. A method has been worked out for controlling that wetness for which concrete block or concrete building units are particularly adaptable. It is necessary, of course, to moisten the wall first, and it is in that point of moistening the wall that we have a chance to control the water content of our stucco. The wall is wetted sufficiently far ahead of the application of the stucco that the block are in a moist condition, but there is no film of water in the pores at the surface of the block. The result is that when the stucco is applied, there is sufficient suction to draw out some of the water in the first coat. That cuts down the water cement ratio in the stucco and takes out the excess water that the plasterer wants for a workable mix. He can get a satisfactory workability with less water by mixing and remixing his material.

On the Home Sweet Home house, we took particular pains to see that the house was wetted down daily with a hose during the curing period. I feel that this accounts for the excellent results. Architects accept the fact that concrete building units make the best possible base for stucco. If the suction in the under coat is properly controlled and each coat in turn kept moist and allowed to cure properly, you will get what amounts to a good job of concrete, which is what you are after. On your finish coat the use of the rough textures that the architects are after now and which are in particular vogue in the east, requires considerable less fussing over the surface than the older float finish coat, and I believe for that reason they are conducive to a good job.

Those things are only to some extent explanations of the standard requirements of the American Concrete Institute on stucco. They are being followed very largely. We are distributing the American Concrete Institute recommended practice to our block manufacturers and to every owner and builder who has anything to do with concrete masonry construction, in so far as it is possible for us to reach him. As a result, we are getting a large number of stucco on concrete block houses in the east. One other reason which I think has some bearing on previous discussions this evening is the fact that it is offering a more economical type of construction. The architects are taking it for a more economical type of construction and are getting that economy. We are getting it there simply because the manufacturers in our district are concerning themselves with turning out in large quantities a plain straightaway utilitarian unit, a large size brother of your every-day common mud brick. It is used for the same purposes for which common brick would be used. Architects and builders are learning that fact and they are using concrete block and finishing with brick veneer or portland cement stucco.

We have cases of the light weight hollow tile being used to the extent of six or seven hundred per man per day. I have talked to men who were laying better than 450 and 500 per day. On a production of that kind we can often show an architect that it is cheaper to build with an 8 in. concrete building unit and a 4 in. brick veneer than to build with 4 in. of lace brick and 4 in. of back up common brick. The extra 4 in. in the wall does not hurt an architect's feelings. There is one point more in my mind that has nothing to do with the subject I am supposed to talk on, but I want to throw out the general question as to why, when you get efflorescence on some of your natural stones and almost invariably on clay brick, why is it more objectionable in a concrete building unit? I have been asked that question a number of times, and I have found so many cases of efflorescence on other materials that I am generally willing to take a chance of walking a man to the nearest window where I can find half a dozen buildings of other materials within view where I can point out the efflorescence either on the natural stone or clay brick and can put the question right back to the man who asked it, and generally it has the desired effect. That is a thought worth carrying out, and it is sometimes very effective.

A collateral case is that, not of craze-cracking, but of the actual cracking in concrete building units. Very frequently buildings built with concrete building units are built on insufficient foundations and a settlement develops with consequent cracks. I suppose about once a week, on an average, I run up against some architect who throws that back at me. My answer is to go down and look at the Biltmore Hotel, one of the finest buildings in the world, or the Woolworth building, one of the best known buildings in the world—the three-story granite piers around the base are every one of them cracked from the sidewalk line to the brick course, and all three faces of the Biltmore Hotel are cracked right straight through; one is granite and one is limestone. There are cast stone jobs in New York, which are not all they should be, but there are plenty of other materials, not concrete, which have developed defects. I sometimes think that we make a mistake in expecting concrete to be an absolutely perfect material. It will do a lot of things that no other material will do, but it it not altogether immune from the action of the laws of physics; it will expand with heat and contract with cold, and if it is not properly handled it will show the effects of those changes.

Instead of apologizing when somebody tells us that so and so's job has developed a slight defect, I think it is perfectly legitimate to come back at them and say that there are a dozen other jobs of other materials that have also developed the same defects; that we know and admit that concrete is not an absolutely perfect building material, but properly used, it will do anything that any other material will do, and frequently a lot of things that other materials will not do, and the 2500 houses built last year of concrete block is evidence that the architects, builders and home owners are beginning to adopt it in the east as a standard commercial structural unit. It is past the experimental stage and is standing on its own feet and is being given full acceptance in all quarters.

Reports of Committees of The American Concrete Institute

Presented at the

Twentieth Annual Convention

February 25-28, 1924.

Report of Committee C-6, on Field Methods.

SUGGESTIONS FOR THE PRODUCTION OF BETTER CONCRETE.

The subject of better concrete is one of vital importance to the industry and which requires the co-operation of the scientists and those engaged in the art of concrete making to bring about its realization.

Better concrete needs to be defined, for it is still generally thought, erroneously, that better concrete is synonymous with increased cost of production. The deficiency of concrete is the extreme variation of its quality and not a consistent inferiority: the potential qualities are not, therefore, fully available.

Better concrete can be defined as a concrete, the properties of which are so nearly constant that they can be fully taken advantage of in design.

It is self-evident that, if concrete of predetermined and constant quality is to obtain, the characteristics of the material must govern in the determination of proportions, and methods must be used to insure constancy of operation throughout the making of concrete.

The production of constant concrete is then based on the following:

(1) A complete knowledge of the concrete making qualities of the available materials.

(2) The application of the scientific principles governing concrete to the determination of the proper combination of the available materials to result in a concrete of the desired quality.

(3) Methods of handling the materials which will maintain the established proportions and insure homogeneity of the concrete to the place of depositing.

It will be of interest to demonstrate:

(1) How far short the present specifications and methods of making concrete fall from meeting the above mentioned requirements.

(2) The fact that in almost every instance a considerable waste of money results from the utter disregard of these rules.

The popular forms of specification in vogue today are to a large extent responsible for the furtherance of the wasteful and inaccurate methods used.

They frequently place restrictions on the gradings of aggregate which are futile and expensive.

They stipulate arbitrary proportions, independent of the nature of the materials, and they leave unmentioned the most important factors which control the quality and cost of concrete.

It is not generally known, but nevertheless a fact, that good concrete can be made with almost any sort of clean, sound and stable aggregate, if sufficient cement is used. The grading of the aggregate, by virtue of the fact that the amount of cement required is dependent upon it, is more a matter of economy than one of regulation. The grading of the entire aggregates (fine and coarse) which proves most economical is that which produces a mixture of particles from fine to coarse having the greatest density or nearly so.

In general, the influence the properties of the constituent of concrete have on the quality of the resulting concrete are as follows:

The cleanliness of the aggregate, its hardness and durability are fundamental requirements.

The relation that the strength of the resulting concrete bears to the proportion, size and grading of the ingredients is:

That the strength of the concrete decreases in a fairly definite relation with an increase in the amount of water used with a given quantity of cement, while the amount of water required to produce concrete of a given plasticity is a function of the quantity and maximum size of the aggregates.

These relations have been sufficiently well established to form the basis for the determination of correct and economical concrete mixtures.

In order to meeet the requirements of these recognized relations the committee offers the suggestions which follow.

The aggregate should be clean, hard and durable. The largest particles should be as large as the type of structure will permit, and the proportion of fine to coarse aggregate such that the density of the total aggregate is a maximum. Sufficient cement and water in a predetermined ratio depending upon the quality of cement and the strength of concrete desired should be added to produce a mixture of proper flowability consistent with the nature of the work.

The carrying out of these suggestions will generally be found to give economical proportions. In some cases the conditions may warrant trials with 5 to 10 per cent less sand than that required to give maximum density to the mixed aggregates.

The method of procedure, therefore, in determining concrete mixtures is as follows:

(1) Select an aggregate the size of the largest particle being as large as the work will permit.

(2) Determine the ratio of fine to coarse which will produce the greatest density of mixed aggregates.

(3) Add cement and water in a predetermined ratio, dependent upon the strength desired, to the aggregate to produce a mixture of such plasticity as is most suitable for proper depositing in the work.

The most important factor affecting the economy of a mixture is that, in a given volume of concrete, the greater the volume of aggregate the smaller the amount of cement and water will it contain.

An economical mixture is one in which a minimum amount of cement is used and, since the ratio of water to cement is definite for a given strength, the dried the concrete, the less cement is required.

It follows then that the most economical concrete of a given quality is that in which the aggregate is graded to a maximum density and mixed as dry as is consistent with the field requirements.

A simple method of determining the economical proportions to meet the above requirements is given below. This method has been tried out and has yielded results which warrant its recommendation by the committee.

Having selected clean, hard and durable aggregates, combine varying proportions of the dry, fine and coarse until a proportion is reached, which gives the densest mixture, i. e., that which weights most per unit of volume.

To this densest mixture add varying amounts of a cement paste (cement and water), premixed in the ratio corresponding to the strength required, until the desired plasticity is obtained.

From the weight of a unit volume of the concrete thus obtained and the weights of the ingredients it contains, derive the proportions and material factors.

The determination requires a very nominal equipment and may be conducted in the field in a very short time, thereby making it possible to make such corrections in the proportions as may be necessary due to changes in the characteristics of the materials or working conditions, thereby maintaining the quality of the concrete uniform and the cost as low as possible.

It is the intention to obtain more definite information during the coming year on the efficiency of field apparatus in overcoming the difficulties encountered in the handling and measuring of aggregate so that there may be placed in the hand of contractors practical, simple methods which will lead to improvements in the quality of concrete and result in a substantial economy.

MANUAL FOR THE DETERMINATION OF PROPORTIONS BY THE MAXIMUM DENSITY OF AGGREGATE AND CEMENT PASTE METHOD.

Determination of ratio of fine to coarse aggregate.

1. Determine the weight of dry, fine and coarse aggregate per cubic foot.*
2. To a given volume of coarse aggregate add in small increments weighed amounts of sand, mixed thoroughly and determine the weight of the mixture per cubic foot for each and every increment of sand. Continue until the weight of the mixture has reached a maximum and shows a decrease in weight by the further addition of sand.

*In all weight determination the A. S. T. M. Standard Specifications are to be followed.

3. Determine from the combination, giving the highest weight per cubic foot, the ratio of sand to stone.

Preparation of cement paste to the proper water cement ratio.

4. Prepare a water cement paste, the total weight of which shall be approximately 35 per cent of the weight of aggregate with which the paste is to be mixed.

$P =$ Weight of paste required
$R =$ Cement water ratio (by volume)
$c =$ Weight of cement required
$w =$ Weight of water required } To make P lbs. of paste

$$w = \frac{62.4 \times P}{62.4 + \frac{94}{R}} \qquad W = \frac{62.4 \times 100}{62.4 \times \frac{94}{R}} = \text{Weight of water contained in 100 lb. of paste}$$

$$c = P - w \qquad C = 100 - W = \text{Weight of cement contained in 100 lb. of paste}$$

Pour the cement into the water, stirring continuously until the whole is thoroughly mixed.

5. To a given weight of the mixture of fine and coarse aggregate mixed in the proportion established as described above, add 1 per cent by weight and mix thoroughly.

6. To the wetted aggregate add small increments of the water and cement paste and mix thoroughly until concrete of the desired consistency results, keeping a record of the weight of paste used.

7. Fill a measure of known capacity with the concrete thus made and determine the weight of concrete therein.

8. From the data obtained from the above described tests, determine the yield factors and the proportions of cement, water, sand and stone.

The following equations are given to show how the yield and proportions can be derived from the test records:

Weight of dry sand per cu. ft. .... $= W_s$
Weight of dry stone per cu. ft. .... $= W_g$
Weight of dry mixed sand and stone per cu. ft. .... $= W_a$
Weight of water and cement paste per cu. ft. .... $= W_p$
Weight of concrete per cu. ft. .... $= W_c$
Sand used in test sample .... $= w_s$
Stone used in test sample .... $= w_g$
Aggregate used in test sample .... $= w_a$
Water and cement paste used in test sample .... $= w_p$
Weight of water added to aggregate .... $= w_w$
Weight of concrete in test sample .... $= w_c$
Weight of concrete in measure .... $= w_c$
Volume of measure used in No. 7 .... $= v$
Volume of batch .... $= V$
Number of test batches in 1 cu. yd .... $= N$

$$w_c = w_a + w_w + w_p$$

$$V = \frac{v - w_c}{w_c}$$

$$N = \frac{27}{V}$$

Weight of water in 100 lb. of water cement paste...... = W
Weight of cement in 100 lb. of water cement paste..... = C
} See Table 1 for determination of these values

	Weight of Material Required to make 1 cu. yd. of Concrete.	Cubic Feet of Materials Required to make 1 cu. yd. of Concrete.	Material Factors P. per yd. of Concrete.	Proportion
Cement paste.....	$=N \times w_p = P_w$	$\frac{P_w}{W_p} = V_p$.	Cement—$\frac{P_w \times C}{100 \times 94 \times 4} = F_c$ in bbl.	1 part
			Water $\frac{P_w \times W}{100 \times 62.4} = F_w$ in cu. ft.	$\frac{F_w + F_w}{4F_c}$ part
Water of absorption	$=N \times w_w = W_w$	$\frac{W_w}{62.4} = V_w$	$V_w = F_w$ in cu. ft.	
Sand............	$=N \times w_s = S_w$	$\frac{S_w}{W_s} = V_s$	$\frac{V_s}{27} = F_s$ in cu. yd.	$\frac{F_s}{4F_c}$ parts
Stone............	$=N \times w_g = G_w$	$\frac{G_w}{W_g} = V_g$	$\frac{V_g}{27} = F_g$ in cu. yd.	$\frac{F_g}{4F_c}$ parts
Aggregate..... ...	$N \times w_a = A_w$	$\frac{A_w}{W_a} = V_a$	$\frac{V_a}{27} = F_a$ in cu. yd.	$\frac{F_a}{4F_c}$ parts

The proportions and material factors are based on dry materials and allowance must be made in the field for the water content of the sand and stone and the bulking of the sand.

The determination of correct and economical proportions is, of course, of no avail unless means are employed in the making of concrete that will insure accuracy and constancy.

The inherent variations of the materials are mentioned below to indicate the necessary changes which must be made to present methods.

Cement: Different brands of cement show wide variation in the strength attained at the period at which the concrete is tested.

Sand: The varying amount of water held by sand causes a variation in the water entering the batch, and the bulking of sand in the presence of water affect to a marked degree the actual amount of sand and water contained in a given volume.

Stone: The variation in the water content of the stone does, as for the sand, affect the quantity of water in the batch.

Sand & Stone: Variations in the grading affect the plasticity of the concrete if the quantity of water is maintained constant, and its strength if the amount of water is changed to maintain a constant plasticity.

Mixing:	All the ingredients which constitute a batch must be mixed into a homogeneous mass, and the length of time the concrete remains in the mixer is vital to uniformity of quality.
Conveying & Depositing:	The transportation of concrete from the mixer to the work must be such as to preclude any segregation.

The making of better concrete is dependent on a number of factors and compensation must be made for the variables enumerated above through the use of specially devised apparatus.

The need of accurate measuring devices for water and aggregates has become apparent to manufacturers of concrete, and a number of devices have been placed on the market which more or less perfectly accomplish the aim sought.

Volumetric measures have been made, which are readily adjusted and so constructed that the material is struck off, thereby furnishing a certain degree of accuracy. These devices are satisfactory for measuring coarse aggregate but of little value in measuring sand, for they do not correct for the bulking or swelling of the sand containing varying amounts of moisture, neither do they compensate for the amount of water contained in the sand and stone which amount may vary from nothing to 30 per cent of the total water required for a batch.

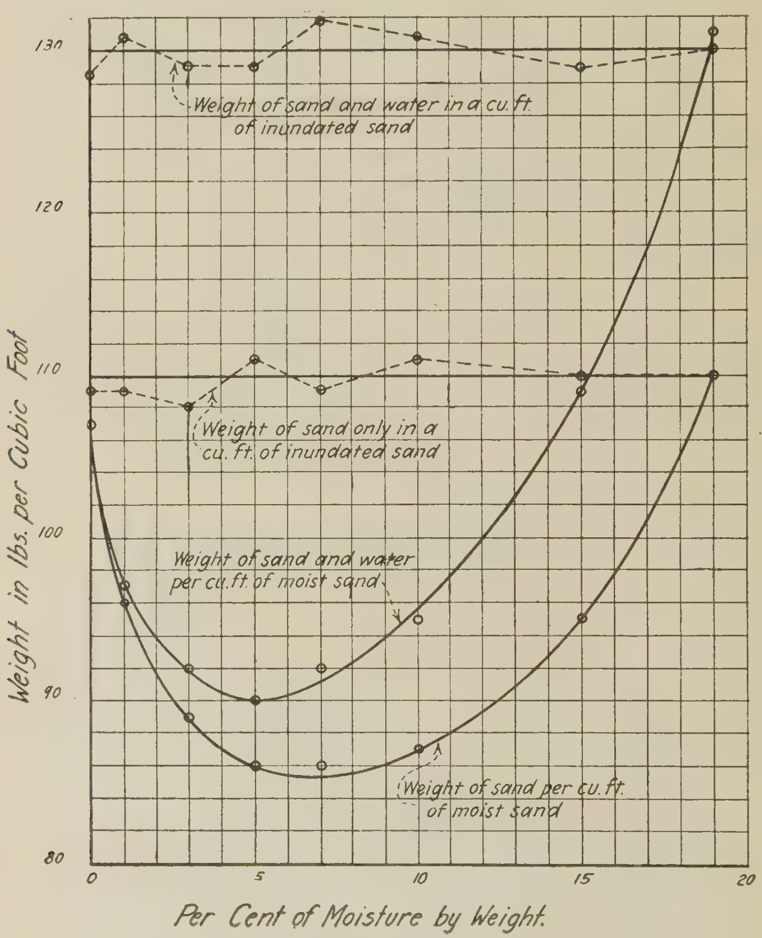
Other devices are constructed so as to determine the quantities of aggregate by weight. Greater accuracy results from this use. The bulking of sand is, of course, compensated for, but the moisture content still remains uncompensated.

Both of these methods render useless and even harmful the use of automatic water measuring devices so constructed as to supply a constant amount of water per batch, for this amount of water is the complement of the variable and indeterminate amount of water held by the aggregate. The general practice, therefore, is to rely on the judgment of the workman at the mixer to add such a quantity of water as he deems proper.

By constant checking of the moisture content of the aggregate and corrections made in the adjustment of the measuring devices mentioned above, fairly accurate results may be obtained.

New devices which automatically compensate for the bulking of sand and the moisture content have been devised which more nearly solve the problem of accurately measuring sand and water than any other known device. They are based on the principle that, if sand containing water in any quantity is dropped in water, the density of the sand thus submerged is approximately the same as that of dry sand and the amount of water necessary to fill the voids of the sand is approximately constant for a given sand.

The accompanying figure is a graph of tests made, showing the variation in density of damp sand measured loosely and of the same sand measured in a submerged condition or inundated.



VARIATION IN DENSITY OF DAMP SAND MEASURED LOOSELY AND INUNDATED.

It is apparent from this graph that by inundating the sand the density of the sand and the water content are made constant or nearly so, and that without any further regulation the accuracy and constancy of the proportions can be maintained from batch to batch.

DISCUSSION

A. N. Talbot.—I feel like offering a word of dissent from the statements put out in this report. It is given as a method of determining the exact quality of the concrete, the most economical relations of the materials and the practical way of settling the amount of cement. It seems to me that the method is based upon such uncertainties that it can not well be used—can not be of general value with the great variety of materials used throughout the country. For one thing, I do not believe, from all the tests of which I have knowledge, that with most materials the greatest density of the mixed aggregate will give the strongest concrete, and if there is a difference in the cost of fine and the coarse concrete aggregate, it may be far from the most economical mixture. The requirement that the largest particles shall be as large as the type of structure will permit, implies that the strength of concrete is dependent upon the size of the largest particles. Within the limit of what would be used in reinforced-concrete work, say between ¾-in. and 2 in., there is no particular difference in the strength that may be obtained by proper proportioning, using workable mixtures. That is shown by many tests. I think the method, too, gives a wrong basis. The most dense aggregate is not necessarily that which gives the most dense and strongest concrete, even if the matter of workability were not taken into consideration. Besides, the method recommended limits the proportions to a single proportion of the fine and the coarse aggregates, when in most cases a considerable variety of proportions may be used to lead to the same results. The method of determining the amount of cement is involved in so many uncertainties that it is likely to carry with it dissatisfaction. I hope the committee will make further tests in the next year, and that they will present more fully the observations on which their conclusions are based.

REPORT OF COMMITTEE E-6 ON DESTRUCTIVE AGENTS AND PROTECTIVE TREATMENTS.

The report of the Committee consisted in the presentation of two papers by members of the Committee, P. J. Freeman and John A. Ferguson, on the subject "Serious Concrete Failures and Their Causes and Remedies." These papers, with the running discussion, follow:

M. M. Upson (*Chairman of Committee E-6*).—No one likes to be the vultures of any particular industry. If we carry on our work as we want to, we necessarily have to bring in the unpleasant odors of bad workmanship, bad material and bad results. The progress report this year will be presented by Mr. Freeman, of Pittsburgh, and Mr. Ferguson, of Pittsburgh. We have endeavored and are endeavoring as we proceed in the presentation of our subject, to pick out one or several unfortunate constructions which show us the horrible example of bad workmanship, bad material, bad methods, bad engineering, and present the fact as concisely as possible with a very detailed explanation of how a recurrence of this experience may be avoided. That is what we are hoping to do for next year. As Chairman of this Committee, and speaking in behalf of the members of this Committee, we are hoping and listening with attentive ears for any suggestions or criticisms which will be helpful in carrying on the aims and ideals of a committee that works under the name by which ours is designated.

P. J. Freeman.—The behavior of concrete under service conditions has recently been receiving more attention than in the past. Engineers have always been too willing to assume that concrete if placed under almost any conditions would always remain perfect and many did not take the trouble to examine their work after a lapse of a number of years.

For the sake of reference attention is called to the symposium held by the American Society for Testing Materials last June and published in the *Proceedings* of that society for 1923. Also the extended discussions which are published in the monthly *Proceedings* of the American Society of Civil Engineers from August, 1923, to January, 1924.

Various opinions are expressed concerning troubles which are being experienced with concrete. Some of the writers place most of the blame on the cement and others would hold the aggregates and methods of handling responsible.

In this paper which will serve to introduce further discussion we will consider two examples of defective concrete and suggest methods for preventing a repetition of the trouble keeping in mind the fact that under present conditions we must use the materials which are economically available for the work.

It is not difficult to find examples of defective concrete, but it is extremely hard to obtain the facts on which to base an opinion. It should

be emphasized that engineers and other should have records of tests of the quality of the materials and also the methods actually used in the construction. The attention to these details will not only tend to eliminate the liability for trouble, but later if an investigation has to be made there will be facts and not theories to work from which will enable the engineer to do better next time.

About 1912 a dam was built partway across a river as the beginning of a large hydro-electric plant, but the project was abandoned until last year when plans were started to complete the development and build the dam entirely across the river. No water had ever passed over the dam and the greater part of it was only subjected to the action of the weather, but during the ten or eleven years the concrete had started to disintegrate so badly that to a casual observer it appeared that the whole structure was doomed. A closer study of the dam checked against the plans and the very complete construction records developed the fact that although the same sand and gravel had been used throughout the condition of the 1: 1½: 3 concrete was like first-class granite, the 1: 2: 4 concrete was satisfactory and the disintegration was confined to the 1: 3: 6 concrete.

The appearance of the disintegrated concrete was similar to that usually encountered in that there was a deficiency in coarse aggregate and the concrete was easily broken out with a pick to a depth of as great as two feet in spots. Underneath this material the concrete was quite hard and many hundreds of feet of cores were drilled for testing purposes. The average strength of this concrete which was mixed 1: 3: 6 was 2166 lb. per sq. in. The compressive strength of the 1: 2: 4 concrete which did not disintegrate was 3400 lb. per sq. in.

It will be seen that insofar as the strength needed for design goes the 1: 3: 6 concrete was more than ample and the 1: 2: 4 concrete was still much stronger, but strength must not be used as the sole guide in selecting the mix for concrete which is to be exposed to the action of the weather, particularly freezing and thawing.

In the report of Committee E-6, on Destructive Agents and Protective Treatments, last year it was repeatedly emphasized that for concrete to be durable, it must be relatively or comparatively impermeable to water. Few people will dispute the truth of the statement, but just why concrete must be impermeable it is an open question in the writer's mind at present. It has been stated that unless the concrete is proof against percolating water, the cement will be dissolved out and thus destroy the concrete. We can appreciate the danger of this condition in concrete placed in certain locations, but in the dam mentioned above the only water which came in contact with the concrete was rain and snow.

This dam is situated in a locality which is sadly lacking in high grade aggregates and recently many tests have been made on the available materials, including the gravel which is obtained at the dam site and which was used in the previous dam built in 1912. When this gravel was subjected to the sodium sulphate test for soundness of rock as recommended

by the Association of State Highway Testing Engineers, after five treatments the softer pieces of gravel had all disintegrated to very fine powder or sand leaving only the rocks which were not affected by the treatment. The method used would probably be about as severe as thirty natural freezings and thawings.

In the examination of many similar cases of disintegration the writer has always been struck by the fact that the coarse aggregate was entirely too low in proportion to the fine aggregate and it was assumed that there never had been a larger amount of coarse material and the placing of the concrete was given a share of the blame. In this particular case it is certain that more than one-fourth of the gravel would readily be disintegrated by frost action in a 1: 3: 6 concrete.

To be sure the disintegration started in the concrete as it was first placed, which was a lean mixture, but we have not been disposed to question the durability of gravel which has existed "through the ages."

In any case the test of time has shown that if the same materials are again placed in the same way, using a mixture as rich as 1: 2: 4, the concrete will not be affected in at least ten years.

Some years ago a dam was built in the far west so high up in the mountains that it was exposed to very severe weather conditions. The mix was 1: 2: 3, the coarse aggregate being a local rock at the dam site which contained large quantities of shaley material. After a short time signs of deterioration appeared and it was given a gunite coating. In a few years the gunite came off, taking a layer of the concrete underneath. Very extensive repairs were necessary and the concrete was readily cut out to a depth of a foot or more. During this process of removing the soft concrete it was observed that whenever any of the local granite had accidentally gotten mixed in, the concrete was entirely satisfactory, but the shaley rock which had been chosen for the work was readily disintegrated by the weathering, as it was structurally unsound.

In these two instances at least it is quite evident that the durability of the materials was not considered. In the case of the first dam, the aggregates were durable in the richer mixes, but in the latter case the aggregates were totally unfit for use in concrete.

More care should be used in the tests for the selection of all materials and careful records kept of all factors in the whole construction project, and by this means it will be possible to develop methods of testing which will more accurately predict the actual service of the materials as well as the concrete itself.

Once the fact is fully recognized that under certain conditions some concrete will disintegrate, we can then start a systematic study of those particular conditions as they now exist and by extended study and discussions such as may follow in this meeting the disturbing factors can be eliminated.

There are many instances in the design of structures where changes could readily be made to eliminate the possibility of water from entering

openings and percolating down until it causes trouble. In some cases the proper use of a waterproofing membrane of some kind will save the expense of using a rich mix throughout and in other cases it is only necessary to change some small details of design at no additional expense.

Some very important work is being done in connection with the admixtures which increase the workability of concrete. If by the addition of some less expensive material than cement we are able to make an impermeable concrete which is durable, we should give such material serious consideration even though the addition of the material may reduce the strength of the concrete below that which would be the case if it were not added. The durability of the concrete should be given prime consideration when it is to be exposed to the action of the elements.

It seems quite probable that by using some of the many treatments which are being developed for application to the surface of finished concrete, that it will be possible to protect some concretes which are not otherwise liable to withstand the effects of weathering and our future discussions should cover experience with such materials under severe service conditions.

G. E. WARREN (*by letter*).—Mr. Freeman has pointed out in a graphic manner some of the causes of difficulties in specific structures. Examination of concrete structures or others involving concrete, which has not fulfilled its proper function, is not a problem which can be solved with any degree of certainty or accuracy. It might be said to be akin to the task of unscrambling an egg.

A great deal of time and money is spent in investigating forms of engineering structures of all kinds, frequently with indifferent or wholly unsatisfactory results.

It appears to me that this discussion could not be closed without emphasizing the greater degree of certainty which is now felt on the part of concrete engineers with respect to predetermining the results they are likely to obtain. While there are many factors which go to make up concreting operations that present unsolved problems or indefinite solutions, the extraordinary work of investigators during the last decade has made possible marked developments in construction practice.

The Bureau of Standards, the Public Works' and State Highway Department laboratories, college experiment stations and many other able investigators have jointly or severally developed improved methods in design of mixtures and control of quality over a variety of conditions and with a variety of materials.

The investigators, the practicing engineers and contractors have applied these recent developments to concreting practices with astonishing results. The possibility of producing a concrete in the field with a reasonable degree of regularity and uniformity is made clear in several papers presented at this meeting, which serve to emphasize the points raised by Mr. Freeman that greater attention to design of mixtures and control of the placing of the concrete will render unnecessary many of the difficulties in examining faulty structures.

JOHN A. FERGUSON.—One of the most essential problems with which this Institute has to deal is that of ferreting out the destructive agencies of concrete. Normally this problem is chiefly concerned with all agencies inimical to strength and permanence of concrete construction after it has been manufactured and placed in the structure. Actually preventive treatments if properly applied will forestall the necessity for protective treatments.

However, since we seem to have sufficient illustration of the destruction of concrete and reinforced-concrete, it becomes necessary to make a complete analysis of the situation. It would seem to be the duty of this committee to assemble the available information that will correctly name the destructive agencies, analyze them for cause and effect and arrange this information in order so that the proper protective treatment may be devised for each destructive agency and then determine upon the application of the appropriate protective treatment.

Obviously preventive measures are to be preferred to protective treatments; both will be found to have their places, however. By far the greater emphasis has been rightly placed upon preventive measures, but protective treatments must be made available for use when preventive measures have not proven effective against unusual conditions or have not been applied.

This particular contribution to the subject is the investigation into disintegrating concrete in the Reed Avenue Viaduct and Arch bridge of Monessen, Pa. This bridge is approximately 510 ft. long and 32 ft. in over all width. The superstructure consisted of floor and sidewalk slabs resting on the longitudinal stringer beams which were in turn supported upon cross beams which rested on concrete columns. The bridge as a whole consisted of a centrally located reinforced-concrete arch of 150 ft. span, flanked on either end by viaduct construction reaching to the small approach abutments.

The bridge was built during the summer and fall of 1912 and winter and spring of 1913. During construction in the winter, the aggregates and water were heated so that the temperature of the concrete going into the forms ranged from 60 deg. to 100 deg. Fahr. The heat used was greater on colder days and when cold weather was anticipated than on warmer days and when warm weather was anticipated just following placement. Whenever cold weather was anticipated than that of the day of placement the temperature of the concrete was measured by the thermometers embedded in a tube in the placed concrete in such a way as to register the temperatures of the concrete for several days or until it had cooled to that of the surrounding air. This took from 24 to 36 hours and sometimes 50 hours in isolated cases when cold weather was not encountered subsequent to placement.

On one portion of the bridge the engineer and contractor had in the early fall been surprised by a freeze of several days' duration without any preventive measures having been taken. Surprisingly enough this concrete showed no disintegration whatever.

In 1920 there was observed that cracking of a lot of the concrete has progressed to such a point that attention was required to determine the safety of the bridge members so disintegrating. The writer made the investigation for the City of Monessen and found that the ends of the longitudinal roadways stringer beams were cracking in numerous places, to-

TABLE 1.—ANALYSIS OF CONCRETE IN MONESSON BRIDGE.

	1	2 *a*	2 *b*	3	4	5	6	7 *a*	7 *b*	8	9	10	11	12
	Beam Bracket above Expansion Plate	Column O P, North Central Part of Cantilever		A-B Span, Corner of Column on North Side	Column 11, North Side, Cantilever Bracket	E-F Span No. 1, South Side E Tower	East Abutment under South Stringer	Cantilever Bracket O P, North Extreme End		North Column O P	South O P Cantilever Bracket	D-E Span No. 1, North Side East Tower	O P Column West Tower South	E-F Span No. 4, North Side under Girder
		3-in. back	1-in. face					Out-side	Inte-rior		Very dirty			
Gravel, 10 mesh	100 {	77.00	49.00	71.20	35.50	47.1	75.00	......	26.50	43.00	47.00	67.00	77.30	70.00
		23.00	51.00	28.80	64.50	52.0	25.00	100.00	73.50	57.00	53.00	33.00	22.70	30.00
ANALYSES OF 10 MESH														
Volatile	11.78	10.00	14.00	16.00	13.46	13.46	11.24	24.56	16.17	12.00	19.30	15.72	14.64	16.10
Insoluble	51.70	55.98	51.50	47.42	48.57	57.62	52.44	41.77	50.00	60.41	53.78	56.30	56.64	53.40
Aluminum Oxide	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Iron Oxide	14.22	12.36	12.00	8.60	13.52	8.66	10.88	10.80	9.88	11.06	9.60	10.74	9.40	11.50
Calcium Oxide	17.64	16.28	16.46	18.76	20.54	15.00	13.04	20.66	13.60	14.16	14.50	15.98	14.22	16.40
Sulfur Trioxide	0.60	0.60	0.78	0.54	0.35	0.30	0.35	0.53	0.43	0.56	1.12	0.70	0.86	0.70
Magnesia and Carbon Dioxide	4.06	4.78	5.26	8.68	3.56	4.56	2.05	1.68	0.00	1.81	1.80	0.56	4.24	1.90
Carbon, free	3.36	2.24	0.95	0.90	0.56	3.00	1.42	1.26	2.98	1.87	0.47	1.54	0.79	1.56
Carbonate	0.40	1.45	1.50	3.00	1.00	2.40	0.45	trace	trace	trace	trace	trace	trace	trace
ANALYSES OF CONCRETE														
Cement	28.22	6.00	13.10	8.64	21.37	12.70	5.21	33.60	16.10	12.90	12.28	8.47	5.15	7.87
Sand	57.46	15.30	30.40	15.52	35.92	33.04	18.07	42.58	40.45	35.68	31.63	21.29	14.64	18.76
Gravel	0.00	77.00	49.00	71.00	35.50	47.10	75.00	0.00	26.50	43.00	47.00	67.00	77.30	70.00
Volatile	11.78	2.30	7.14	4.60	8.68	7.12	2.81	24.56	11.88	6.84	10.17	5.19	3.32	4.83
Undetermined	2.54	....	0.36	0.24	....	0.04	....	.....	5.08	1.58	....	....	....	....
Showing amount of SO_3 if due entirely to cement, deducting 1.75 giving excess														
(*a*)	2.11	2.25	3.00	1.80	1.09	3.04	1.66	1.60	2.00	2.40	4.80	2.70	3.75	2.65
(*b*)	0.36	0.50	1.25	0.05	....	1.29	....	....	0.25	0.65	3.05	0.95	2.00	0.90

(*a*) Shows SO_3 total.

(*b*) Shows excess over what should be in cement or (*a*)—1.75.

gether with the exposed ends of the cantilever brackets supporting the sidewalk fascia stringer, on the bottom of the roadway slab at the expansion joints and practically all the bottom of the sidewalk slab and numerous other positions of less importance.

Every surface showing this disintegration was more or less covered with a white efflorescence. The sidewalk slab which had been concreted after April 6, 1913, was practically covered. The efflorescence on the underside of these members occurred where percolating or seeping water came through the concrete. The materials of the efflorescence were often heavy enough to form stalactites a half-inch long.

Upon tapping the sides of the longitudinal floor-beams where disintegration of the concrete was apparent a hollow sound was given out.

A very complete analysis of the proportions of the several kinds of concrete found in typical parts of the bridge. This analysis, as would be expected, bore out the visual appearance of the concrete in the sample.

A very interesting chemical analysis of the several kinds of concrete that seemed to typify the grades of the good, bad and in between good and bad concrete. The results of these analyses are tabulated in Table 1.

Professor Stevens, of the Carnegie Institute, made some observations in connection with the analyses which are here quoted:

"It is evident that little information can be obtained from the sulfuric acid content as it may be and presumably is due to sulfating from atmosphere, the river gravel carries mechanically, appreciable amounts.

"If it were entirely due to this the samples containing the least cement should show highest sulfuric anhydride.

"Referring to the table:

"Sample 11 showing 2.00 per cent great excess of SO_3 has a gravel and sand content of 88.29 per cent, which is high.

"Sample 9, with 3.05 per cent SO_3 excess has a gravel and sand content 78.60.

"Sample 6, however, with a deficiency of SO_3 has a gravel content of 93.07. The large amount of very coarse gravel would not carry as much as SO_3 as the fine.

"However, samples 1-7a-7b having highest sand content are not high in SO_3.

"It may be noted that no sample high in cement carries an excess of SO_3.

"The free lime and carbonate as given in table do not consistently follow the composition of the concrete, and are merely relative results."

Samples of the good, indifferent and bad concrete were submitted to the Pittsburgh Testing Laboratory.

A water extraction test was carried out by immersing the sample in distilled water for the purpose of determining the nature of the soluble substances in the concrete. These analyses are tabulated in summarized form as follows:

Analysis of Water Leach	Sample 1	Sample 2	Sample 3
		Parts per 100,000	
Silica	5.38	Trace	4.5
Aluminum and iron oxides	.50	Trace	Trace
Calcium carbonate	9.63	66.65	17.4
Magnesium carbonate	Trace	Trace	Trace
Hydrated calcium sulphate	.40	1.36	47.00
Magnesium sulphate	Trace	Trace	Trace
Sodium sulphate	13.80	Trace	32.2
Sodium chloride	2.64	4.70	13.3
Total solids extracted	33.7	75.5	125.0

Sample 1—Large piece of concrete, good.

Sample 2—Small piece of concrete from expansion joint was taken from disintegrated concrete.

Sample 3—Flat piece of concrete from bottom of sidewalks was taken from very disintegrated concrete.

	Sample 4 Per cent	Sample 5 Per cent
Sand and gravel	67.58	84.26
Soluble silica	2.10	.66
Calcium oxide	12.26	6.18
Alumina and iron oxide	2.72	1.12
Loss on ignition	15.70	8.36
Sulphuric anhydride	0.36	0.36

The analysis was made of samples of concrete, but to ascertain the proportions of possible destructive chemicals to be found in concrete corresponding with samples for the Water Leach Test as given in the foregoing table. Sample 4 in the above table is therefore taken from the same concrete as Sample 2, and Sample 5 is taken from the same concrete as Sample 3.

A sample of the white surface deposit was analyzed as follows:

	Per cent
Silica	15.00
Aluminum oxide	13.20
Iron oxide	2.80
Calcium oxide	32.00
Magnesium oxide	Trace
Sulphuric anhydride	7.28
Carbon dioxide	Present
Loss on ignition	29.97
Chloride	Trace

Approximate Combinations.

	Per cent
Sulphate of lime (hydrated)	15.60
Cement	43.00
Clay	14.65
Sand	14.65
Combined water and carbon oxide.....	26.75

The conclusions drawn from these analysis were as follows:

The analysis of the white surface deposit shows that it is composed of hydrated cement, which forms rather more than half of it, the remainder being about equal parts of hydrated sulphate of lime and fine clay and siliceous material. The substance which gives it its white color is the sulphate of lime, which is present in far higher ratio in the cement than is normal.

The ratio of sulphuric anhydride to cement is about normal in the expansion joint samples of concrete. It is about double the normal ratio in the sidewalk slab samples.

The water extracts show that the sidewalk slab yields to water a high percentage of calcium sulphate and a medium amount of calcium carbonate, while the extract from the expansion joint sample is just the reverse. The water extract from the large lump of concrete yields no calcium sulphate and only a moderate amount of calcium carbonate.

The damaged concrete was all removed and replaced by good concrete.

A beam typical of the concrete that was to be left in place was cut out, lowered 125 ft. to a truck transported 60 miles to Pittsburgh and tested in the testing station of the United States Bureau of Standards. The load was applied at the third points to develop the diagonal tension as well as the transverse strength. It failed simultaneously in compression of concrete, and tension of reinforcement and was almost at the point of failure from diagonal tension.

The load carried was such that the beam was considered to have a factor of safety of approximately 4 for a live-load of a 30-ton interurban street car. This was taken as evidence that the structure was adequate when and after the work of repair was completed.

The significance of the chemical analysis is worthy of note. Undoubtedly in this case the destructive agency was the presence of the acid radical of SO_3 which became H_2SO_4 from contact with percolating water. Every portion of the bridge showing disintegration had been subjected to percolating water for long periods of time, and consequently in cold weather frost action undoubtedly hastened the disintegration. The latter being thus one of the destructive agencies.

The protective remedy applied was to replace all unsatisfactory concrete by new concrete made under more modern methods and to waterproof the bridge roadway under the pavement by means of a bitumastic coating over the roadway slab. It was not thought necessary to waterproof the sidewalk slab because this was entirely removed and replaced.

This is a brief summary of a much more detailed investigation. J. Campbell of J. N. Chester, Engineers, was engineer in charge for his firm. Professor S. S. Stevens, Carnegie Institute of Technology and Pittsburgh Testing Laboratory, made the analyses. John F. Casey Contracting Company, S. Fuller, Engineer and Manager, executed the work. Mr. Johns was City Engineer, and the writer was consultant on the plan and execution of the work.

The principal lesson to be learned from this repair and investigation is that it is not sufficient to properly proportion the materials and produce concrete of as dry consistency as may be used and fill all parts of the forms, and be properly consolidated around the reinforcement. Good, strong, permanent concrete can only be made after adding to the foregoing sufficient care in transporting and placing to insure concrete as thoroughly mixed in place as when it leaves the mixer.

There is a difference between disintegration and decomposition. Disintegration is a mechanical breaking up of the mass into smaller units, while decomposition is a chemical breaking up, disunion or rearrangement of the materials. The product of decomposition differs from the original. Decomposition and disintegration may occur simultaneously or separately.

In investigating concrete chemically to ascertain whether the trouble is due to disintegration or decomposition, it is not sufficient to analyze chemically the cement sediments as the several constituents of the cement even though found to be present, are not conclusive evidence. It is doubtful if the chemist can ascertain whether the chemical combinations of the cement had taken place in proper form or whether they had been broken up or whether they had recombined into different molecular units. However, where a portion of the cement was dissolved and carried along in water and deposited where evaporated, it is quite probable that the chemist can ascertain the combinations with sufficient accuracy so that practical conclusions may be drawn.

Disintegration may be caused by freezing and thawing while in a saturated condition. Water expands 10 per cent in freezing. The force of expansion is practically irresistible, breaking up in time the strongest rock. Expansion and contraction due to changes in volume of the several materials will result in disintegration and the result is hastened by differential expansion and contraction when the coefficients of the several matérials differ among themselves. The natural process of all rocks exposed to weathering conditions is one of disintegration or disintegration and decomposition. Even the best of concrete is not entirely exempt from this process. The historic examples giving the longest life are located in climates not subject to great periods of changes in temperature. However, it is perfectly true that well-made concrete will amply justify its existence as regards permanence and freedom from disintegration.

Decomposition of concrete is almost always brought about in the presence of water. Water percolating through concrete will invariably dissolve all lime not permanently combined with acid radical, and some of the

latter forms are slowly soluble. Free lime, either hydrated or dehydrated, is quite soluble. Silicates, if soluble at all, are soluble when free lime is present without the hydroxyl. Silicates are not rapidly soluble under the most favorable conditions and in cement that is properly sintered and the silicate reaction satisfied there is practically no solubility.

The problem of producing concrete that is proof against decomposition resolves itself into:

(a) Production of sell-burned cement that will be chemically permanent in the presence of percolating water or water containing acid or alkaline chemicals.

(b) Concrete that is as dense and impremeable as possible and where conditions are such that the water may carry an acid radical in solution in appreciable quantities, such concrete should be protected from contact with or percolation of water.

Concrete containing SO_3 is not in danger unless water is present to form H_2SO_4. The danger zone of serious decomposition from SO_3 does not begin until the ratio of SO_3 exceeds one part per thousand.

The foregoing suggests that permeability rather than porosity or density may be the more valuable criterion of resistance, and that the aim should be to produce impermeability rather than low porosity of absorption. However, it is difficult at first to visualize concrete that may be porous, have a high absorption, and low density and that will at the same time be of low permeability. Practical consideration would lead one to expect all of these to go hand in hand. But the greatest of them is impermeability. To whatever degree any or each of the others is present it is necessary to realize that the function of impermeability must be realized as far as possible.

Thus the necessity for progress in investigation and execution must be apparent. The fact that there are a great many very satisfactory structures, should not blind us to the necessity for a better knowledge of the processes involved in the manufacture of cement and of concrete. The approximations of yesterday must be of the accuracies of today, backed up by detail attention to matters once thought trivial.

A MEMBER.—Did you have to do anything with the arch ring?

MR. FERGUSON.—Nothing at all.

A MEMBER.—Was the same aggregate used in the arch ring?

MR. FERGUSON.—All through the bridge. Considerable care was used in casting the arch, I understand, owing to the fact that the contractor was new to arch span work and was very careful. There was a one line cable way which was not movable. The concrete was hoisted onto the cableway, carried to the center of the bridge, dropped down there and apparently shoveled from there into the beams and girders, and the further away you get from this cableway, the worse the concrete gets; the nearer you were to the cableway, the richer the concrete and the more dependable it was at the ends of the beams where apparently this concrete had been

deposited, but at the center of the span it was shoveled to the beam and allowed to slope either way; at the ends of the beams was where the trouble was found.

A MEMBER.—Was the same mix used?

MR. FERGUSON.—Yes, throughout the bridge from beginning to end, even the foundations. There were other items in connection with the construction of the bridge, but I do not know whether it is wise to expose them at this time. One of them is that under some of these girders on the abutment we found about four bushels of pure sand and there was apparently no cement or aggregate present at all. That is just an indication of several other things we found.

F. C. WIGHT.—I do not get just what the speaker gives as the explanation of the trouble.

MR. FERGUSON.—I have been unable to decide that question myself, except based on the idea that the contractor was far more careful in placing the concrete in some places than others, and where the concrete is not porous the water is not seeping through it, and while the condition of the concrete is about the same as any good concrete in other parts of the bridge, it is possibly not ideal concrete.

MR. WIGHT.—Then the excess SO_3 that you think is in the aggregate is affected only where seepage occurred?

MR. FERGUSON.—Only where seepage has occurred. Thinking of it that way, the SO_3 became sulphuric acid when the water came in and dissolved such free lime as was soluble and carried it away, leaving the freezing and thawing to do the rest of the work.

A MEMBER.—You say the SO_3 was supposed to have come in with the sand from the Ohio River?

MR. FERGUSON.—That is the best estimate we can make from the records of the bridge and the analysis.

A MEMBER.—Were analyses made of the Ohio River sand?

MR. FERGUSON.—We made no further analyses of the Ohio River sand to back that up. Our investigations stopped at the bridge because they did not give me the opportunity to go any further.

LANGDON PEARSE.—You speak of SO_3; do you mean that you found sulphate in the aggregate?

MR. FERGUSON.—The chemist found SO_3.

MR. PEARSE.—I do not see how he could find that.

MR. FERGUSON.—He did not find the sulphates, he only thought they were present because he found SO_3, and apparently, according to his explanation, it was impossible to find what combinations SO_3 had taken.

MR. PEARSE.—Certain waters there are known to be acid, I think the Allegheny River in particular, and the Monongahela River water. Chemists of the geological survey will tell you that some of those streams are acid from the mine waste, not the steel mills, and those waters will attack limestone and create sulphates, and in the investigations we made to determine the cause of breaking down, we decided that the active thing would

be the sulphates present which, in themselves, might be weak, but with the water on them might give trouble.

MR. FERGUSON.—Quite likely that is the explanation; not being a chemist and this being the first time I hit this thing, I have not gone quite as far as you did.

D. A. ABRAMS.—I examined this bridge in October, 1920, shortly after it had been closed to traffic and while reconstruction operations were in progress. Only an hour or two was spent on the structure and I had no opportunity to go into the records of the work as Mr. Ferguson probably has done. It seemed to me from a hasty examination of the structure and knowing absolutely nothing about the circumstances under which the work was done that sloppy concrete mixtures and the method of distributing, which put a premium on the use of plenty of water in order to get the concrete out to the edges, is a sufficient explanation of the failure of the deck portions of this bridge.

I also found that the concrete contained a large percentage of coal up to a ¼-in. size. No doubt this coal was brought in with the aggregate and the farther away the concrete was shoved, the more coal came to the surface. The combination of an excess of mixing water, the coal and freezing weather conditions seems to be ample explanation for the results found.

MR. FERGUSON.—I think that is a good explanation; I believe Mr. Abrams' conclusions are correct, too.

C. M. CHAPMAN.—If this bridge was in the steel district where furnace gases are large in quantity, there may be an explanation for this SO_3 present, or sulphur and some compound. It has been shown by experiments in connection with the bleaching out of sulphates and sulphur in the soil, that in some of the manufacturing districts the amount of sulphur brought down by precipitation, collected out of the air in the sulphur fumes of burning coal, amounts to as high as 40 lb. per acre per year. I do not know this bridge; the surface might have approximated a fraction of an acre, and if the district uses a great deal of coal, we might have as much as 40 lb. of sulphur precipitated on that bridge.

MR. FERGUSON.—Not only the area of the bridge in this case might be counted in as regards the number of pounds of sulphur per year in the water which ran down on the bridge, but the roadway for some distance on one side sloped up for half a mile, maybe, and the water all ran down that roadway and onto the bridge and across it. Now Monessen is quite a steel town; the bridge, however, is on the outskirts and a little one side of the prevailing wind, so that the gases from the blast furnaces would only occasionally reach the territory of the bridge.

REPORT OF COMMITTEE G-4, ON NOMENCLATURE.

A Discussion on the Definition of Concrete, by W. A. Slater, Chairman of the Committee.

There has been a demand for a public discussion of the Institute's definition of concrete. In fact, I have been informed by the Secretary that the Board of Direction took action providing for such a discussion. The committee would have been glad to have this discussion before this, but the custom of the Institute has been to discourage discussion of nomenclature reports on the floor of the convention.

The definition of concrete which, except for minor amendments, has been a standard of the Institute for five years is

> *Concrete*—A compound of gravel, broken rock, or other aggregate, bound together by means of hydraulic cement, coal tar, asphaltum, or other cementing materials. Generally (always in the specifications of the American Concrete Institute) when a qualifying term is not used, portland cement concrete is understood.

There has been a movement to restrict the definition in such a way that even with the qualifying term present the Institute does not recognize the right of other industries to use the word concrete in connection with a material which does not contain portland cement.

The word "concrete," with qualifying words, is in fairly common use, to denote other than portland cement concrete. This fact is recognized in the definitions found in the Standard dictionary, the International dictionary, the Century dictionary, and the latest edition (11th) of the Encyclopaedia Britannica. The special committee of the American Society of Civil Engineers on Road Materials (See Am. S. C. E. Proceedings, December, 1914, 3011), prepared a definition of bituminous concrete pavements.

It is not uncommon in technical articles to use the term asphaltic concrete in exact acordance with the definition under discussion. In *Engineering News-Record*, July 19, 1924, there is a letter which uses this term. It is true that the writer of the letter was probably interested commercially in the matter, but so also are the majority of those who have written me during the past year in favor of the exclusion of all but the portland cement interests from the use of the term "concrete" in any form. The authors of articles in the paper *Public Roads*, published by the Bureau of Public Roads, presumably have no proprietary interest in promoting asphaltic concrete, and yet the term asphaltic concrete is used in the issue of June, 1918, p. 23 and 28. In his paper "Highway Research in Illinois" in the February *Proceedings* of the American Society of Civil Engineers, Clifford Older, Chief Highway Engineer for Illinois, uses the expression "Asphaltic concrete surfacing on a concrete base" to describe a type of pavement. He thus takes the committee's stand precisely in rec-

ognizing that without a qualifying term the word concrete means portland cement concrete, but that other kinds of concrete need the qualifying term and are entitled to the use of the term "concrete." The above references found without special search are sufficient to show that authors who have occasion to consider concrete having other than a portland cement binder recognize the correctness of applying to it the term "concrete" with a qualifying term.

Of the members who support the idea that the terms "asphaltic concrete," "bituminous concrete," etc., should not be used, I should like to ask, "What word is proposed to take the place of the word concrete?" Thirty years ago asphaltic concrete was common and was known as concrete. Some of the asphaltic concrete is still in existence. If it is not now asphaltic concrete, when did it cease being asphaltic concrete and what is it now?

A letter from John R. Nichols has stated the position of the Committee so clearly that it is here quoted.

"June 23, 1923.

"Prof. W. A. Slater, Chairman
Committee G-4,
Bureau of Standards,
Washington, D. C.

Dear Sir:

I have a circular letter from the Portland Cement Association in regard to the definition of concrete. You are doubtless familiar with the contents of this circular letter.

Instead of urging, however, that the definition of concrete adopted by the joint committee be substituted for that given as Definition 25 of the American Concrete Institute, it is my opinion on the contrary that the Institute's definition is better.

The word 'concrete' was used for tar concrete long before portland cement concrete was common, and at that time the word 'concrete,' without qualifying term, was generally understood to mean tar concrete. In the last twenty-five years, however, the use of portland cement concrete has so enormously increased that the word 'concrete,' without qualifying term, is now generally understood to mean portland cement concrete. This does not signify, however, that tar concrete is not concrete, and I consider that it is not at all proper for the Institute to attempt to exclude concretes with other than portland cement binder from the use of the term.

I conceive that the definition adopted by the joint committee was intended to apply to the use of the word in the committee's report and was given in order that it might be clear that the report did not apply other than with portland cement binder. If the Institute adopts the joint committee's definition, it will be construed as an effort not to make clear the actual meaning of the word 'concrete' but simply as stating a limited meaning to be applied in the Institute's own publi-

cations. The Institute, in the last analysis, cannot alter the meaning of the word 'concrete'; only common usage or good usage can determine this meaning. It is my opinion that the definition quoted by the circular letter as that of the Institute is clearer because it not only gives the true, broad meaning of the word but also indicates its usual limited meaning when used without qualifying term.

Yours very truly,

(signed) JOHN R. NICHOLS."

I have not yet mentioned the thing which in connection with this definition seems to me most important. Whatever we do, whether we accept the definition as the committee has given it, or insist that if the material does not contain portland cement it is not concrete of any kind, the result will be the same; everybody will know that concrete without a qualifying term is portland cement concrete. So far as the definition is concerned, then, this seems to be a tempest in a teapot, but another matter is involved which seems to me to be of more importance than all those mentioned, and that is the standing of the Institute. Is this a trade association or a technical society? I want it to be considered that it is a technical society and that simply because the portland cement industry has 99.9 per cent of the usage of the word "concrete" we are not going to try to force other industries, which have some more limited use of the term, to recognize that nothing is concrete that does not have portland cement in it. If we were to take such an attitude we would put ourselves in the position that we are a trade association rather than a technical society.

It has been suggested in correspondence that we omit the words "Generally (always in the specifications of the American Concrete Institute)" making the definition to read, in closing, "When a qualifying term is not used, portland cement concrete is understood." That would express the present practice in the vast majority of cases and as one member of the committee I would not oppose the change. I believe, however, that the definition is better, as it stands at present, and more likely to be given weight even in a court of law, because it is a careful statement. It makes the position of the Institute clear and does not assume the prerogative of legislating outside of its own sphere.

REPORT OF COMMITTEE P-5, ON FIRE RESISTANCE OF CONCRETE BUILDING UNITS.

FIRE RESISTANCE OF CONCRETE BUILDING BLOCK.*

The fire resistance of building materials is a matter of extreme importance to the community at large. Our huge annual fire losses (63 million dollars in residences alone) are causing increasing concern, and the conviction is rapidly growing that in the public interest and for public safety, the use of permanent, fire-resistant materials ought to be encouraged—especially as their relative cost today is so little in excess of less permanent materials. Present legislative restrictions on their full and free use ought to be examined and modified wherever present day experience shows this to be possible.

In this particular, the work of the Building Code Committee of the Department of Commerce is worthy of special mention and it is to be hoped that the adoption of this code with its reasonable regulations for the reduction of the thickness of masonry walls of brick, concrete or tile, and its provision for fire stopping and chimney construction, will soon become universal and lead the way to a greater use of fire-resistant construction.

Non-Flammable and Fire-Resistive Material.—The old idea that any incombustible material was necessarily fire-resistive has been the cause of many disastrous failures. It is recognized today that many materials that will not burn are failures as fire retardants. Materials vary so much in thermal conductivity, in rate of expansion when heated and in strength after heating; factors that are of paramount importance in fire resistance. The fire-resistive qualities of a building material bear no relation to their strength or conductivity. In building materials like gypsum, steel, clay tile, concrete block, lumber, cast iron and stone are found some that are high in fire resistance and low in strength and vice versa and even those that are high in both vary a great deal in thermal conductivity, rate of expansion and contraction and other factors that effect their stability. Therefore each building material must be examined, tested and rated, on its merits from the structural and fire-resistive point of view.

Extensive Use of Concrete Building Block.—The rapid increase in the use of hollow concrete building block and building tile in the construction of residences and garages, and its use as a backup for brick veneer in large buildings, is drawing a great deal of attention to this economical and popular building unit.

* The illustrations noted in this report are not published herewith.

The incomplete statistics that have been gathered for the years 1922 and 1923 indicate that at least 350 million concrete block and building tile were sold during each year, and it is probable that an accurate count would prove nearer to 400 million. The smaller figure is the equivalent of over four billion common brick as laid in the wall, (more than half the annual output), and indicates the universal acceptance of concrete block as a first-class structural material. Its economy leads many builders to use it insead of frame construction as by its use permanent, fire resisting construction is obtained at a cost that only exceeds frame by a very small margin.

In the Building Code drafted by the Chamber of Commerce Committee, concrete block is specified for residence construction on a parity with hollow clay tile or brick, and other similar building materials. This approval of concrete block as a standard building material will draw further attention to its excellent merits and will undoubtedly increase its use. Building codes of most cities permit its use but in the minds of many engineers, doubts are still unsettled as to its fire resistance owing to the fact that in some cases that come under their observation, concrete block walls have not resisted fire satisfactorily.

In view of the fact that many individual cases have shown negative results, although concrete block as a rule have shown high fire resistance, it seems well to analyze the fire resistive qualities of concrete block and point out those factors which affect its strength under fire.

Fire Resistance of Concrete Varies According to Aggregate.—The fire-resistive qualities of solid monolithic concrete are very well known and have been very completely covered in the column tests carried out at the Underwriters' Laboratories in 1917-1919. These tests on monolithic concrete and concrete protected steel columns have established the fact that the fire resistance of concrete varies considerably according to composition of the aggregate and therefore in analyzing the fire resistance of any concrete block the kinds of aggregate as well as the method of manufacture and form of the block must always be taken into account.

Problems of Fire Resistance.—From a fire insurance engineer's standpoint, a wall may fail at one of three critical points, the first is the passage of flame through the wall. Second, the transmission to the unexposed face of the wall of heat exceeding 300 deg. will cause timber or other inflammable material that touches or rests on the wall to char and often will ignite. Third, the loss of stability under load when heated may cause such structural weakness in the wall that although it remains intact during the fire, it is of no further value as a load-bearing structure.

The fire resistance of solid monolithic concrete has been well established. But in determining the fire resistance of hollow building block of concrete or terra cotta, we are faced with additional factors, the most important being the internal stresses around the hollow cores.

When heat is applied to one side of a hollow concrete block or clay

tile the exposed surface tends to expand. This expansion is not uniform through the block and on the unexposed face at the beginning of the fire there is no expansion at all. When the full thickness of the interior wall of the block has become heated the shearing stress, which up to that time was taken up by the full vertical area of the block, is concentrated on the thin cross webs and although the total stress per block is the same the stress per square inch borne by the webs is much greater than that previously borne by the exposed face. There is therefore a tendency to crack at this point and this accounts for the large number of block found to be cracked when removed from the fire.

At the 1923 convention the committee recommended in its progress report that certain changes be made in the program, in order to investigate time of web cracking or other failure and to determine what protection is afforded by stucco or plaster; and therefore in accordance with permission given by the Institute several changes were made in the original program. Panels A, G, and H had been tested before the presentation of the 1923 report. Panels B and D were the next to be tested; Panel B was built of two cell block with rectangular cores and Panel D with building tile 5 x 8 x 12 in. as described in Appendix A of 1923 report, pp. 355 to 359 of *Proceedings*, 1923. One-half of each of these panels was covered on the exposed side with portland cement stucco before being tested.

Panels C, E and F as originally planned were omitted from the test program and in their place a series of tests was made in a specially contrived small furnace in order to determine if possible the cause of the web cracking.

In the 1923 report it was observed that web cracking was apparently the critical point on which the fire resistance of concrete block must be judged, as in other respects the fire resistance was highly satisfactory. Accordingly attention was concentrated on this feature.

The tests on panels B and D one of which contained standard size concrete block and the other contained concrete building tile 5 x 8 x 12 in. showed that a protective coating of portland cement stucco had a very marked influence in reducing the amount of cracking especially in Panel B in which practically all the unstuccoed block were cracked while only 30 per cent of the stucco protected block cracked. The stucco block also resisted the transmission of heat for about one hour longer.

It was therefore thought possible that web cracking might take place during the period in which the block was raised from 200 deg. to 300 deg. on the outside face.

Before building up another large panel it was deemed advisable to discover if possible if these cracks were caused by rapid expansion at the start of the test or to excessive expansion at higher temperatures.

Some trial tests were therefore arranged in a small furnace and a series of small panels about 36 in. square listed as panels O, P-1, P-2, P-3, P-4, P-5, R, S, T, U, V, W. In building the blocks into the frames of this furnace an ingenious arrangement was devised whereby the interior of the block could be observed during the test. The lower two courses of some of

the panels were laid in as usual but the upper courses were not completed and the top course and the half filled third course were filled in with 4 in. gypsum blocks. This enabled us to look into the interior of the block during the progress of the fire test.

The tests in this type of panel were inconclusive; in every case web-cracks were observed in periods ranging from 10 to 25 minutes. The top block was free to expand in all directions like a block in an unrestrained wall owing to the low strength of the gypsum block around it. This upper block cracked at the same time as the others and thus disposed of the argument that in unrestrained walls less cracking should be expected and this conclusion agrees with the observations made by S. H. Ingberg of the U. S. Bureau of Standards, whose fire tests on brick walls showed that better results under fire test were gained from the restrained panels than the unrestrained panels.

It seems to be well established therefore that web cracking, if any, may be expected within 30 min. from the start of the test and that block which have not cracked by that time will remain intact.

It was also observed that the results obtained in these small panels were far less satisfactory than in the standard large panels and therefore a final test panel K was built using block that had been prepared for panels E and F, standard 1 : 5 block steam cured and air cured respectively. The results from this test confirmed the previous tests showing the added protection afforded by portland cement stucco.

Details of these tests follow:

DESCRIPTION OF PANEL.

Fire Endurance Test Panel B.

Panel B was built of two-core concrete blocks manufactured of 1 : 5 mix. It was necessary to chip the ends from a few of the blocks in order that they should fit into the 10 ft. opening in the brick panel. The mortar was used in laying up the blocks in a 1 : 3 portland cement with 10 per cent by weight of the cement of hydrated lime similar to that used in Panels H, A and G.

The blocks were installed as specified for Panel H. The north half of Panel B was stuccoed with portland cement stucco as follows:

The blocks were thoroughly dampened before application of the first or scratch coat.

The following aggregates were procured from a supply of material at the Laboratories which was bought in the open market. The stucco was applied by plasterers obtained from the McNulty Bros., Plastering Contractors, Chicago.

Each batch consisted of the following:

2 pails fine lake sand (44 lb. each)	88 lb.
1 pail screened Torpedo sand (44 lb. each)	44 "
1 pail portland cement	50 "
5 lb. hydrated lime	

The stucco was applied in regular three-coat work. The finish coat had the sand screened in order to eliminate all coarse material.

The scratch coat was applied about ¼ in. in thickness, thoroughly scratched, then brown coat was applied same day scratched, making a (thickness of about ¾ in., then a damp canvas was applied next day, making the stucco about 1 in. in thickness, the damp canvas was applied over same and allowed to remain in position without further dampening for about ten days before removal.

The blocks in Panel B were 59 days old, when tested; they were laid up in the panel 34 days and the north half of the panel was stuccoed 27 days before test. The panel was allowed to season in a room where the temperatures are kept well above the freezing point.

No cracks were apparent in either the blocks or in the stucco previous to test although a few hair cracks were observed in the mortar joints.

The test was continuous until the temperature on the unexposed face of the unstuccoed blocks averaged 300 deg. F. when the panel was removed from the furnace for further observation.

The general appearance of sample before test is shown by Figs. 1 and 2.*

Observations During Test Exposed Face.—The fire was semi-luminous for short intervals during the first 75 min. due to adjusting the furnace fire, the flames were less luminous than usual during this period but fire was luminous during the remainder of the test. The fire had a tendency to swerve from north to south side during the first 75 min. During the latter part of the test the fire had a tendency to impinge for short intervals on the south or unstuccoed section of the panel. The fire in general was more severe on the unstuccoed section of the panel during the first 165 min. due to considerable difficulty in regulating furnace fire during this period.

The average furnace temperatures and the average temperatures on the stuccoed and unstuccoed section and on the unexposed face of the panel are shown by Fig. 7.

Portland cement stucco finish began to separate and bulge away from the concrete blocks about 3 feet above the sill at 16 min., this separation gradually increased up and down from this point and at 20 min. the stucco had bulged away about ½ in. in the area where first noted. The separation had extended from sill to about the middle of the central vertical edge of the sample at 27 min. when the greatest bulge was about 1¾ in. This bulge gradually increased to about 3 in. at 35 min. showing very little change until about 56 min. when an irregular crack about 3 ft. long appeared in the bulged area. At this time the bulge had apparently separated the stucco from the blocks and extended about one-half way across the lower half of the panel. At 75 min. several more irregular cracks appeared in the bulged area. At 120 min. the stucco in the bulged and

*Illustrations are not included in this preprint.

cracked area was insecure and apparently was dehydrated. At 183 min. a section about 2½ sq. ft. of the stucco in the bulged area fell from position and after this period the bulge in the stucco began to decrease. At 209 min. or 3 hr. and 29 min. when the furnace fire was extinguished and the panel was removed from the furnace another section of the stucco fell from position.

Condition of the stucco about 18 hr. after removal from the furnace chamber is shown by Figs. 3 and 4.

No apparent spalling or cracking was observed in the exposed face of the blocks during test.

Observations on Unexposed Face of Panel During Test.—At the beginning of the test small hair cracks were apparent in the mortar joints between the blocks. These hair cracks became more apparent during the first 15 min. of the test and appeared at several other points which were not noted before or during the first 15 min. No cracks were observed in the blocks during the test.

Moisture appeared at the joints on south of the area opposite the unstuccoed section during the first 15 min. This moisture increased in volume at these points during the first 20 min. when moisture appeared at the joints on north side of the area opposite the stuccoed section. Steam appeared near the center of the top of the panel at 32 min. and appeared along the north and south edges of the panel at 40 min. At 60 min. the moisture decreased and at 75 min. all the moisture had disappeared on the south half at the mortar joints. At this period the moisture on the south side began to disappear and at 130 min. nearly all the moisture had disappeared from the unexposed face of the panel. At 180 min. only occasional jets of steam were apparent along the edge of the panel.

The appearance of the unexposed face of panel after fire endurance test is shown by Fig. 5.

The temperatures as indicated by thermometers attached to unexposed face of panel is shown by Fig. 7.

The panel bulged toward the fire a maximum of 2¼ in. at center at 210 min. The bulge was gradual and uniform from the beginning of the test until the panel was withdrawn at 210 min. The bulge in the panel gradually decreased after the test and at 18 hr. after the panel was withdrawn from the furnace chamber the maximum bulge was 1 3/16 in.

Observation After Fire Endurance Test Panel B.—The general condition of Panel B after test is shown by Fig. 3 and the condition of the stucco in the damaged area is shown by Fig. 4.

No apparent cracking was observed in the exposed face of the blocks after test before sample was dismantled and then each individual block was carefully examined.

The stucco was completely dehydrated especially in the damaged area where the heat penetrated through the cracks and behind the bulged area. The stucco finish was covered with a great number of very fine irregular

cracks and larger cracks corresponding to the size of the blocks behind it as shown by Fig. 4.

The bond between the stucco and the blocks in the undamaged area was secure and remained in position in the majority of cases after the blocks were removed from the panel.

The exposed face of the unstuccoed blocks was dehydrated to a depth of about 5/16 in., and this calcined area fell from position when exposed to the air for about three weeks.

Condition of Blocks After Removal From Panel B.—There were fifty-nine blocks unstuccoed, fifty-nine blocks stuccoed and seventeen in the middle of the panel partially stuccoed and unstuccoed.

Unstuccoed Section—Twenty-seven of the unstuccoed blocks had three webs adjacent to the inner face of the exposed shell cracked and also a vertical crack in the exposed shell at one side of the central web. These cracks are indicated in block in Fig. 6. Eleven blocks were broken or cracked in removing from the panel. Nine blocks had three webs cracked adjacent to the inner face of the exposed shell. Three blocks had two webs and one vertical crack in the exposed shell. Three blocks were undamaged. The remainder of the blocks in the unstuccoed section were cracked in various combinations as specified above. No cracks were apparent in the unexposed shells or webs. The cracking in the shells and webs was more general in the lower half of this section than in the upper half. It was only possible to observe the various cracks as specified above under very favorable light conditions, and in a great many cases were not apparent under casual observation. It was possible to handle these blocks after removal from panel without danger of the cracked sections falling apart. The greater number of these blocks were shipped by truck to Lewis Institute without any serious damage after removal from the panel for further test and observation.

Stuccoed Section—There were 59 blocks in the stuccoed section and 38 were apparently undamaged and in good condition. Eleven were broken in removing same from panel. Three had one end web cracked. Four had vertical crack in exposed shell. The remainder of the blocks in this section were cracked in various combinations of the above shell and web cracks. The stucco apparently protected the blocks to a considerable extent.

Partially Stuccoed Blocks—There were 17 of these blocks. Six blocks were undamaged. Three had two shell cracks. Two had two web cracks and the remainder were cracked in various combinations of above.

Fire Endurance Test Panel D.

The blocks were installed similar to previous panels having south half of panel stuccoed as follows:

The blocks were thoroughly dampened before application of the scratch coat.

The following aggregates were procured from a supply of material at the Laboratories which was bought in the open market. The stucco was

applied by plasterers obtained from the McNulty Bros., Plastering Contractors, Chicago.

The stucco was applied in regular three-coat work. The finish coat and the sand screened in order to eliminate coarse material.

Stucco Mix Panel D

Scratch and Brown Coat

2 pails fine lake sand (41½ lb. each)	83 lb.
1 pail screen torpedo sand	52½ "
1 pail portland cement	48 "
5 lb. hydrated lime	
32 lb. water	

Mixed thoroughly dry and wet.

Scratch coat applied on Feb. 9 at 12 a. m. then scratched ⅜ to ½ in. in thickness.

Brown coat applied next morning.

Finish coat applied 6 days later. Brown coat was dampened before application of finish coat.

Mix

1 pail torpedo sand	58½ lb.
2 pails fine lake sand	93 "
1 pail portland cement	47½ "
hydrated lime	5½ "
water	35

The standard test equipment was used in this test. The test was continued until the temperatures on the unexposed face of the unstuccoed blocks reached 300 deg. F. when the furnace fire was extinguished and the panel immediately removed and allowed to cool for further observation.

The panel was tested March 16, 1923.

The panel was 11 ft. high and 9 ft. 8 in. wide.

A thorough horizontal crack was apparent in the mortar joints extending through the stucco about 42 in. above the sill. The cause of this crack was not apparent. Location of this crack is shown by Fig. 8.

The general appearance of panel before test is shown by Figs. 8 and 9.

Observations During Fire Test Exposed Face.—The furnace fire was more excessive during the first hour on the stuccoed or south half of the panel due to trouble in adjusting the furnace fire.. The fire was uniformly distributed during the remainder of the test. Slight impinging of flame on sample was noted for short periods during test.

Temperatures in furnace chamber during test is shown by Fig. 15.

A slight separation was noted in the joint between the stucco and blocks at 45 min. This separation became more apparent at 75 min. and increased slightly until about 140 min. indicating very little change during the latter part of the test. The size and location of this separation and general condition of the stucco after test is shown by Figs. 11, 12, 13

and 14. The original horizontal crack which was noted during test showed no apparent change during test.

No other spalling or cracking was apparent during test. Furnace fire was extinguished and sample withdrawn from the furnace at 3 hr. and 36 min. when the temperature on the unstuccoed section of the blocks on unexposed face averaged 300 deg. F. The temperatures on the inner face of the exposed shell and in the center of the hollow spaces and on the unexposed face of the panel are shown by Fig. 15.

Observations on Unexposed Face of Panel D During Test.—On the unexposed face of the panel moisture appeared on the area opposite the unstuccoed section at 16 min. and at 30 min. the moisture appeared generally at horizontal joints in the upper half of panel and increased in volume until about 55 min. when the moisture at the joints began to diminish until at about 175 min. the moisture had practically all disappeared.

At 35 min. steam issued at the junction between the panel and the brick frame near the middle of the north edge and at 50 min. steam appeared along the south edge, at 55 min. steam issued from a mortar joint, near the thermocouple marked No. 2 in the stuccoed area. Steam appeared at several joints at 70 min. and gradually increased in volume until about 180 min. when the volume gradually diminished from all joints. At 15 min. an irregular mortar joint crack appeared in the upper north corner and at the same time a horizontal mortar joint crack appeared about 25 in. above the sill; at 17 min. another horizontal crack appeared about 48 in. above the original horizontal crack which was noted before test. An irregular mortar joint crack appeared in the lower north corner at 50 min. At 135 min. a vertical crack appeared in the blocks about 42 in. from the north edge and extended 96 in. above the sill. Two other vertical cracks appeared in the blocks at 135 min. and 145 min. and joined the other long vertical crack forming a fork with two prongs and a long handle. The location of these cracks are indicated by chalk marks on Fig. 14.

The panel bulged toward fire a maximum of 2 1/16 in. at the middle in a uniform bulge at 160 min. remaining constant until 190 min. and decreasing to 2 in. when the sample was withdrawn from the furnace at 216 min. (3 hr. 36 min). The maximum bulge was 15/16 in. toward fire at 18 hours after test.

The temperatures on the unexposed face of panel are shown by Fig. 15.

Appearance of unexposed face of panel after test is shown by Fig. 14.

Condition of Blocks After the Fire Endurance Test.—The unstuccoed blocks were practically all cracked as shown by (1) Fig. 13. It was difficult to remove samè from panel without the exposed cracked section separating from the remaining section.

The blocks in the stuccoed section were cracked generally as shown by (2) Fig. 13 except that several blocks had an additional crack in either of the end webs and a few blocks had a vertical crack through each shell

and the middle web dividing the blocks into two sections. It was possible to handle these blocks without same falling apart.

Auxiliary Fire Endurance Tests.—As a result of the foregoing tests, it was apparent that web cracking was an inherent characteristic of the blocks tested regardless of aggregate, method of curing or shape of the block. So the next step in the investigation was to determine at what period web cracking appeared and in order to determine this feature a series of tests was made, the blocks being similar to those already tested. The tests to be run for varying periods.

These tests were made in the small brick furnace as shown by Figs. 20 and 21 with structural steel framing and fire brick lining. The furnace was supplied with city gas for fuel, mixed with air supplied by a blower. The combustion chamber was 33 in. wide, 22 in. deep and 35 in. high at the center of the arched roof.

The small furnace provided with the triplet panels as shown by Figs. 20 and 21 was employed in these tests. The blocks were laid up similar to the previous samples allowing about one week to season.

Results of these small panel tests established definitely that web cracking occurred during the first 30 min. of the tests and definitely disposed of the theory that restraint panels gave severer conditions than the unrestrained—as the top blocks which in some panels were surrounded with gypsum blocks of like strength, allowing the concrete to move freely—cracked just as quickly as the others.

From the results of the first group of these small samples it was apparent that general web and shell cracking occurred early in the test, and from results of tests of the stuccoed blocks in panels B and D there was some indication that ¾ in. portland cement stucco decreased the general web cracking to some extent but as these tests were not conclusive in order to come to a definite conclusion on this feature it was decided that small triplet panels should be constructed and same to be stuccoed with regular ¾ in. portland cement stucco.

The exposed shells in the blocks used in the following tests were about 1⅜ in. thick and when the ¾ in. stucco was applied made a total thickness of 2⅛ in.

It was apparent from the results of test of these stuccoed panels that blocks with shells 1⅜ in. thickness and stuccoed with ¾ in. portland cement stucco showed general web cracking similar to the unstuccoed blocks with shells about 1⅞ in. thick. It was observed that type B standard blocks which had shells about 2 in. in thickness gave more favorable results; when ¾ in. stucco was applied to this shell the thickness of the exposed shell averaged about 2¾ in.

It was then decided to run one more large panel (Panel K) completely covered with portland cement stucco in order to check up on the results of the previous stuccoed panels.

Fire Endurance Test Panel K.—Panel K was constructed with blocks of the following types as described in Table I, Appendix 1. The lower

four rows of Type E-3, next four of Type E-2, next four of Type E-3, and the remainder of Type F-2 blocks.

The panel was constructed similar to the previous samples, was 10 ft. wide and 11 ft. high.

The sample was stuccoed with two coats ¾ in. portland cement, interior stucco, similar to that applied to Panels B and D, except that the stucco was applied in two coats and no damp canvas was used as described in previous samples.

The sample was about 35 days old when tested and the blocks were about one year old; they had been stored inside, protecting them from extreme weather conditions.

Appearance of sample before test is shown by Figs. 41, 42 and 43.

Observations During Test Exposed Face.—Fire was semi-luminous during the greater part of the test, having a tendency to fluctuate in intensity from the north to the south side during the first part of the test becoming more evenly distributed at 20 min. and remaining fairly luminous during the remainder of the test. The fire had a tendency to impinge on the sample during the test.

The temperatures in the furnace chamber in the hollow spaces and on the unexposed face of the sample is shown by Fig. 46.

No spalling or cracking was apparent on the face of the sample during test.

Observations of Unexposed Face.—Cracks appeared in the mortar joints in the lower north corner at 10 min. and in upper north corner at 20 minutes. No cracking or spalling was apparent in the blocks proper during the test.

The panel bulged ¾ in. toward the fire at 20 min., 1⅜ in. at 60 min., 2⅛ in. at 120 min., 2 9/16 in. at 180 min.. and was 1⅜ in. after the panel was allowed to cool.

Appearance of unexposed face of sample after test is shown by Fig. 43.

The temperatures on the inner face of the exposed side in the hollow spaces and on the unexposed face at the middle of the panel, are shown by Fig. 46.

Examination of Blocks After Test.—The blocks were removed from the panel and carefully examined under very favorable light conditions, and the following results were noted.

It was possible to handle all blocks after removal from panel without same separating, due to the dove-tailing of the coarse aggregate in the walls of the webs and shells. Cracks were difficult to discover under the most favorable conditions and it is possible that other cracks may have been present in the blocks than those indicated in the following summary.

These cracks as specified below became more apparent after the blocks were tapped with a hammer or the blocks were used with ordinary handling, indicating that although apparently insignificant at first, when jarred the blocks had a tendency to separate and fall apart.

Type F-2 Blocks.

About 33 per cent of blocks Type F-2 examined were uncracked, 20 per cent had exposed vertical shell and all webs cracked, 20 per cent had two webs cracked, the remainder had various combination of the above cracks.

Type F-3 Blocks.

About 50 per cent of Type F-3 blocks had one vertical exposed shell cracked and all webs 25 per cent had only the exposed shell cracked and the remainder had various combinations of one exposed shell and one or more webs cracked.

Type E-2 Blocks.

About 40 per cent of the Type E-2 blocks examined had only one vertical crack in the middle exposed shell, 38 per cent had the exposed shell and two adjacent end web cracks, and about 23 per cent had the vertical exposed shell and all webs cracked, the remainder were cracked in combinations of the above.

Type E-3 Blocks.

About 90 per cent of Type E-3 blocks had all webs cracked and one vertical crack in the exposed face. A few blocks had two vertical cracks in the exposed face in addition to the above web cracks, the remaining blocks had cracks of various combinations of the above and a few had one or two webs uncracked.

CONCLUSION.

The results of the tests are highly encouraging for they have shown that the fire resistance of the standard concrete block tested is far greater than had been supposed.

The test program was not completed until December 28 and the results are still being studied by the Underwriters Laboratories—who will shortly issue a report on them to their Council.

Prior to the publication of this report it would be premature to make a final report on the proper fire resistance ratings of hollow concrete block as in comparison with other building materials.

The following conclusions, however, may be drawn at this time from the results of the tests:

1. Walls of moderate lengths, properly constructed with regard to piers and buttresses of well-made, 8-in. concrete block can be relied upon to resist the passage of flame in fires of ordinary intensity for a period exceeding five hours without spalling or cracking of the surface—and if exposed to fire hose stream during that period will not collapse. The impact of falling beams is not likely to overturn the wall.

2. A temperature of 300 deg. Fahr. on the unexposed face of block in an 8-in. wall may be expected at periods ranging from two hours and

thirty minutes to three hours and thirty minutes, according to the form of the block and the aggregate used.

3. The bond between portland cement stucco and concrete block is sufficient to cause the stucco to adhere when exposed to furnace temperatures and can be relied upon to delay reaching the 300 deg. point for one hour and also protect the block in large measure from the splitting or cracking observed in some panels of the unstuccoed block.

4. The exposure to flame for three hours or more causes a deterioration of the exposed face and loss of strength in the block up to as much as 30 or 40 per cent when not covered with stucco. But the fire tested block regain a good deal of their strength after the fire and in no case does the strength of the unit go below the usual allowable loading of city building codes.

5. Web cracking may be expected in a large proportion of hollow concrete building units exposed to fire. This cracking occurs early in the fire, usually before 30 min. has elapsed.

The question remains unanswered as to whether the structural stability of the wall has become seriously impaired when web cracking has occurred. Although this point is very important it cannot be regarded as a critical one as long as there is no comparative data on brick and clay tile walls. The report of the Bureau of Standards in the recent fire tests in brick walls points out that compression tests made of masonry before and after fire tests, while indicating adequate strength for most of the brick, indicated in some cases such loss of strength in the fire test, that, taken in connection with the deflections obtaining at the end of the test might prejudice ability to carry working load.

Tests made on block taken out of the panel after the fire test show a loss of strength varying between 25 and 40 per cent so that it might be reasonable to assume that block carrying one thousand pounds per square inch on the gross area before the test would be good for at least 600 lb. per sq. in. afterwards. Concrete block walls are practically never used for heavy loads. Building codes usually allow a maximum load of 70 to 100 lb. per sq. in. on concrete block walls. The practice almost universally adopted is to use a structural frame when heavy loads are to be carried. In residences the load on the wall very seldom exceeds 40 lb. per sq. in. In buildings of this class there should be, therefore, an ample factor of safety even after a severe fire if there is not more than 50 per cent of cracked block.

The remarkable recovery in the strength of block after removal from the fire (especially on those block which are first subjected to an absorption test) would indicate that there is no concealed fracture in any of the block, as even a recovery in strength of the concrete itself would not result in an equal recovery in the strength of the block if there was concealed internal cracking.

The effect of this web cracking upon the ability of the wall to safely bear its load during or after the fire exposure period is dependent upon several variables including the height of the wall, the live load which it carries, and whether or not this load is uniformly or eccentrically applied with respect to the wall thickness.

The importance of web cracking in its influence on the load bearing qualities of the wall could not be determined with the Laboratories facilities at our disposal, and it is very desirable that further investigations be made on this point. There can be no doubt that when a large percentage of blocks are cracked the strength of the wall is thereby reduced, but we could not determine whether the strength had been reduced to a point where the factor of safety had been encroached upon.

Building codes usually call for a load not exceeding 100 lb. per sq. in. on walls of concrete block. The Standards of the American Concrete Institute for medium load bearing walls call for block 750 lb. per sq. in., and as far as pure resistance to compression is concerned the walls have such a large factor of safety that even if the whole load were carried after a fire on the inner half of the block a wall in a normal building loaded up to the limit allowed by the code would have strength enough to carry the load and inasmuch as the walls showed no serious face cracking or sign of disintegration and were uninjured by the falling beam except for the hole cut at the point of impact it would seem that concrete block walls were amply strong for the purpose for which they are commonly used.

Consideration of stability of wall for use in rebuilding seems to us to be of similar moment, but we think it may be fairly argued that this assembly must be at least as stable as brick or clay tile walls.

Take, for intance, the common brick wall laid in lime mortar. Even if such a wall remains uncracked the intense heat of a severe fire would dehydrate the mortar entirely* and you have as a result a wall consisting of brick resting in a bed of material having little more strength than sand. If the wall is laid in portland cement mortar it may be a little stronger but it is obvious that the customary methods of laying a brick wall in cement mortar could not result in producing such a strong mortar as the mortar or concrete in the ordinary concrete block. Moreover, brick are not usually true to shape, their irregularities being taken up by the mortar. The concrete block is true and square and is laid with a much thinner joint; and therefore, assuming that the mortar in a wall laid in cement mortar and a wall of concrete block are affected to the same degree, the strength of the concrete wall must obviously be greater after a fire than that of the brick wall laid in cement mortar.

Clay tile has not only the disadvantage of irregularity in shape and thick mortar joints above referred to, but it is also subject to cracking of the webs. Clay tile also are ordinarily laid in lime mortar while concrete block are laid in cement or lime—cement mortar.

---

*Bureau of Standards Report of Fire Tests on Brick Walls states "There was loss in strength of lime mortar approaching disintegration which must be considered where masonry strength is concerned."

6. The method of manufacture and curing, which had a marked bearing on the compressive strength of the block had practically no effect on its fire resistance. In this connection it is interesting to notice that block in Panel B which had the highest strength in compression had a higher percentage of cracking under fire than Panel A. Panel D in which the quality of concrete was the highest had nearly every block cracked during the test, however, these units had very thin shells.

7. Although there was a slight variation in the performance of blocks of different form made on different machines it may be stated that for practical purposes as regards one piece hollow units the form of the block had very little effect on its fire resistance.

8. The kind of aggregates used had practically no bearing on its fire resistance except in the case of coarse aggregate of highly siliceous material, which seemed to be less fire resistant than other material.

No detail analysis was made of the gravels to determine the composition of finer particles as distinct from coarser particles. It is generally found the so-called calcareous aggregates have a fairly high percentage of siliceous material and in such cases the finer material is usually siliceous while the larger pebbles are calcareous. It is noticeable that all of the fine aggregates used had a high percentage of siliceous material. (See tests on aggregate samples 6416 and 6417.)

It is, therefore, the expectation of the committee that 8-inch concrete block walls of good quality block not containing highly siliceous aggregate and passing the A. C. I. tentative standards Class B will be recognized in Underwriters' Laboratories' report as a satisfactory fire resistant in panel enclosures, and other non-load bearing walls and also in lightly loaded walls.

When block are protected from direct contact with fire by brick veneer, portland cement stucco, or plaster, their fire resistance is considerably increased and they can then be recommended for heavier load bearing.

The tests have all been made on 8-inch walls of hollow concrete block and in order to complete a study of this subject it is highly desirable that a further program of tests should be laid out to include:

(a) 12 in. walls of hollow block
(b) 8 in. walls of 2-piece block

In the light of the tests already made it is probable that each of these constructions will show higher fire resistance than those tested in the present series.

It is also essential that further tests be made on 8-inch walls under load in order to determine what effect the web cracking has on the stability and load bearing qualities of the walls.

The committee desires to place on record their deep appreciation of the many courtesies and splendid co-operation of the Underwriters' Laboratories and particularly to mention the untiring zeal and energy of M. J. O'Brien, of the Underwriters' Laboratories; E. W. Dienhart of the Portland Cement Association, and Stanton Walker, of the Structural Materials Research Laboratory, who handled all the detail work of the test program and contributed so materially to its success.

LESLIE H. ALLEN, *Chairman.*
HARVEY WHIPPLE, *Secretary.*

TABLE I.—STRENGTH TESTS OF BLOCK BEFORE AND AFTER FIRE TEST.

Tests of three-oval-core block.

Methods of Testing.

1. Tested in accordance with recommendations of the American Concrete Institute.
2. Dried to constant weight.
3. Tested wet after immersion in water for 48 hours.
4. Immersed in water 10 days, dried in laboratory 3 days.
5. Frozen and thawed 5 times; block thawed by immersion in water at room temperature.

Panel.	Date of Fire Test.	Absorption, per cent by weight.		Compressive Strength, lb. per sq. in.								
				Before Fire Test.					After Fire Test.			
		Before Test.	After Test.	Test No. 1.	Test No. 2.	Test No. 3.	Test No. 4.	Test No. 5.	Test No. 1.	Test No. 2.	Test No. 4.	Test No. 5.
A..........	12–28–22	5.3	8.5	1250	1430	1080	1900	1950	1390	990	1540	1560
		5.7	5.7	1010	1730	1080	1570	1690	1170	770	1340	1690
		5.1	8.0	1240	1580	1220	2040	2010	1440	1070	1440	1520
		5.6	8.1	1010	....	1130	....	2300	1490	840	1350	1660
Average.	....	5.4	7.6	1130	1580	1130	1840	1990	1370	920	1420	1610
Date tested..	....	(12–4 –22)	(1–25 –23)	(12–14 –22)	(2–1 –23)	(1–16 –23)	(1-16 –23)	(2–19 –23)	(1–25 –23)	(1–19 –23)	(1–18 –23)	(2–16 –23)
G-1........	12–29–22	5.4	3.1	1155	....	....	....	....	1070	750	880	....
		5.7	3.2	1185	....	....	....	....	890	700	1060	....
		5.3	Bro e	1220	....	....	....	....	710	....	...	....
		5.7	while	1155	....	....	....	..	....	....	740	....
		5.8	hand-	1080	....	....	....	....	....	....	....	....
			ling									
Average.	....	5.6	...	1160	....	....	....	....	980	725	870	....
Date tested..	....	(12–4 –22)	...	(12–14 –22)	...	...	...	...	(1–25 –23)	(1–19 –23)	(1–18 –23)	
G-2........	12–29–22	5.3	6.1	1350	2170	1150	....	2070	1490	600	950	....
		4.3	6.7	1820	1690	1360	....	1850	1360	570	1070	....
		4.9	6.1	1720	2060	1160	....	2160	900	810	1460	....
		4.0	...	1600	....	....	....	2110	....	....	....	....
		4 6	...	....	....	....	....	....	....	....	....	....
Average.	....	4.6	6.3	1620	1970	1220	....	2050	1250	660	1160	....
Date tested..	....	(12–4 –22)	(1–25 –23)	(12–14 –22)	(2–1 -23)	(1–16 –23)	....	(2–19 –22)	(1–25 –23)	(1–19 –23)	(1–18 –23)	
G-3........	12–29–22	4.9	...	1590	2580	1770	....	Over	....	....	1990	....
		3.7	...	1950	2250	1950	....	ca-	....	....	880	....
		4.0	...	1870	2420	1710	....	pacity.	....	....	....	....
		4.1	...	1460	....	2070	....	(2500)	....	....	...	....
		3.8	...	1830	....	....	....	....	...	....	....	....
Average.	....	4 1	...	1740	2420	1880	....	....	....	....	1440	....
Date tested..	....	(12–4 –22)		(12–14 –22)	(2–1 –23)	(1–16 –23)	....	....	....	....	(1–18 –23)	....
H-1........	12–27–22	5.7	11.9	1220	....	790	....	1400	....	730	....	....
		5.9	17.1	1000	....	510	....	1430	....	680	....	....
		5.9	11.8	990	....	....	....	....	....	....	....	....
		5.3	7.5	920	....	....	....	....	....	....	....	....
		6.0	7.6	920	....	....	....	....	....	....	....	....
		...	5 6	....	....	....	....	....	....	....	....	....
		...	7.1	....	....	....	....	....	....	....	....	....
		...	7.8	....	....	....	....	....	....	....	....	....
		...	9.1	....	....	....	....	....	....	....	....	....
		...	6.8	....	....	....	....	....	....	....	....	....
		...	7.8	....	....	....	....	....	....	....	....	....
Average.	....	5.8	9.1	1010	....	650	....	1420	....	700	....	....
Date tested..	....	(12–4 –22)	(1–2 –23)	(12–14 –22)	....	(1–16 –23)	....	(2–19 –23)	....	(12–30 –22)	....	....

TABLE I.—STRENGTH TESTS OF BLOCK BEFORE AND AFTER FIRE TEST.—*Continued.*

Panel.	Date of Fire Test.	Absorption, per cent by weight.		Compressive Strength, lb. per sq. in.								
				Before Fire Test.					After Fire Test.			
		Before Test.	After Test.	Test No. 1.	Test No. 2.	Test No. 3.	Test No. 4.	Test No. 5.	Test No. 6.	Test No. 2.	Test No. 4.	Test No. 5.
H-2.........	12–29–22	11.2	17.5	960	1030	790	800	....	....	580	....	....
		12.6	17.5	920	1250	710	1200	....	....	530	....	....
		11.7	17.6	980	1260	800	1310	....	....	600	....	....
		12.1	16.6	950	....	....	....	....	....	570	...	....
		12.0	16.5	1020	....	....	....	....	....	(12–30 –22)	....	....
		....	17.6	....	....	....	....	....	....	580	....	....
		...	18.1	....	....	....	....	....	....	530	....	....
		....	18.2	....	....	....	....	....	....	600	....	....
		....	17.5	....	....	....	....	....	....	600	....	....
		....	....	....	....	....	....	....	....	(1–6 –23)	....	....
		....	....	....	....	....	....	....	....	720	....	....
		....	....	....	....	....	....	....	....	560	....	....
		....	....	....	....	....	....	....	....	690	....	....
Average.	....	11.9	17.5	970	1180	770	1100	....	....	660	....	....
Date tested..	....	(12–4 –22)	(1–2 –23)	(12–14 –22)	(2–1 –23)	(1–16 –23)	(1–16 –23)	...	....	(1–9 –23)	....	....
H-4.........	12–27–22	5.5	....	1360	1500	850	1580	1480	....	780	820	....
		5.6	....	1110	1350	970	1470	1150	....	810	980	....
		5.8	....	980	1040	990	2210	1410	....	610	910	....
		6.7	....	910	1750	1020	....	1710	....	750	900	....
		6.5	....	870	....	....	....	....	....	(12–30 –22)	....	....
		...	....	....	....	....	....	....	....	820	....	....
		...	....	....	....	....	....	....	....	790	....	....
		...	....	....	....	....	....	....	....	610	....	....
		...	....	....	....	....	....	....	....	(1–6 –23)	....	....
		...	....	....	....	....	....	....	....	980	....	....
		...	....	....	....	....	....	....	....	1110	....	....
		...	....	....	....	....	....	....	....	1280	....	....
Average.	....	6.0	....	1070	1410	960	1750	1440	....	1120	....	....
Date tested..	....	(12–4 –22)	....	(12–14 –22)	(2–1 –23)	(1–16 –23)	(1–16 –23)	(2–19 –23)	....	(1–8 –23)	....	....
H-H.......	12–27–22	7.8	11.1	1010	....	790	....	1010	....	530	....	....
		6.4	10.5	980	....	510	....	....	....	550	....	....
		6.9	11.0	1040	....	....	....	....	....	540	....	...
		7.4	11.7	990	....	....	...	....	....	(12–30 –22)	...	....
		7.2	11.2	1100	....	....	....	....	....	650	....	....
		...	11.6	....	....	....	....	....	....	630	....	....
		...	12.1	....	....	....	....	....	....	...	....	....
Average.	....	7.1	11.3	1020	....	650	....	1010	....	640	....	....
Date tested..	....	(12–4 –22)	(1–2 –23)	(12–14 –22)	....	(1–16 –23)	....	(1–16 –23)	....	(1–6 –23)	....	....

TABLE II.—STRENGTH TESTS OF CONCRETE BLOCK AFTER FIRE TEST FROM PANEL "B".

Strength tests of two-square core block.

Lot No.	Face of	Compressive Strength of Block, lb. per sq. in. of Gross Area.					
		Before Fire Test	After Fire Test.				
			Air of Laboratory.		Air of Laboratory 8 days, Remainder		Air of Laboratory 8 days. Remainder Out of Doors.
			7 days.*	28 days.*	21 days.*	28 days.*	28 days.*
6580........	Plain...	2350†	1420	1150	....	1670	1540‡
		2350†	1610	1350	....	1870	1490‡
		2350†	1370	1000	....	....	....
Average.	....	2350	....	....	....	....	1520
		1860	930	....	....	....	....
		1780	....	....	....	....	....
		1700	....	....	....	....	....
Average.	....	1780§	1330	1170	....	1770	....
Date tested..	....	1-18-23	2-20-23	3-13-23	....	3-13-23	3-13-23
6581........	Stucco..	....	2080	1430	1550	1660	1390
		....	1640	1220	1610	1590	1430
		....	1840	1490	1560	1580	1260
		....	1900	1400	1480	1530	1120
Average.	....	....	1860	1380	1550	1590	1300
Date tested..	....	....	2-20-23	3-13-23	3-6-23	3-13-23	3-13-23

* Time after fire test.
† Over capacity of 300,000-lb. testing machine. Calculated on basis of 300,000-lb. strength tests made after absorption tests
‡ Half block.
§ Tested as received.

TABLE III.—STRENGTH TESTS OF CONCRETE TILE AFTER FIRE TESTS FROM PANEL "D".

Strength tests of two-cell 5 by 8 by 12-in. concrete tile.

Face of Panel.	Compressive Strength of Tile, lb. per sq. in. of Gross Area.	
	Before Fire Test.	After Fire Test.
Plain....	1670	810
	1790	540*
	1710	1050*
	1800	1000*
	....	750*
Average....	1740	830
Stucceod....	....	1130
		1020
		1150
		960
		700*
		760*
		1150
Average....	....	980
Date tested....	3-6-23	3-30-23

* Part of tile tested; wall exposed to fire broke off on removal from panel; partition and outer wall intact.

TABLE IV.—CHEMICAL ANALYSES OF AGGREGATES.

Each value is the average of two determinations.

Lot No.	Silica (SiO_2)	Iron Oxide (Fe_2O_3)	Aluminum Oxide (Al_2O_3)	Calcium Oxide (CaO)	Magnesium Oxide (MgO)	Sulphuric Anhydride (SO_3)	Loss on Ignition.	Free Iron (Fe)	Moisture.	Approximate Combustible Matter, per cent.
6411	24.97	7.08*	10.45	4.67	1.02	0.61	46.75	3.45	0.53	47
6412	93.32	2.88	2.00	0.11	0.53	none	0.89	none	....	..
6413	8.51	1.12	1.27	45.78	3.70	0.03	39.06	none	....	..
6414	96.00	1 45	1.42	0.03	0.68	none	0.51	none	....	..
6415	37.50	0.90*	10.61	39.26	4.25	0.99	4.10	1.18	....	..
6416	20.94	1.14	1.76	24.62	16.32	none	35.23	none	....	..
6417	43.76	1.23	3.81	16 90	10.94	none	23.17	none	....	..
6418	38.92	0.26*	10.66	42.71	4.70	0.20	0.82	0.29	....	..

TABLE V.—EFFECT OF AGE ON THE STRENGTH OF CONCRETE BUILDING BLOCK AFTER EXPOSURE TO FIRE.*

Three-oval-core block stored in moist room tested at different ages after fire test.
Compressive strengtn in lb. per sq. in. of gross area.

Lot No.	Panel.	Date of Fire Test.	Date Placed in Moist Room.	Compressive Strength of Block After Storage in Moist Room for			
				7 days.	28 days.	3-mos.	1 year.
6493A	A	12-28-22	1-22-23	880	1380	1290	1880
				930	1360	1210	1950
				1100	1220	1375	1790
Average..				970	1320	1290	1870
Date tested..				1-29-23	2-19-23	4-22-23	1-22-24

* These block are from the same shipment as those listed under Lot. No. 6455

DISCUSSION

Leslie H. Allen.—The report of Committee P-5 is a continuation of the report submitted last year, which described the starting of the program of tests authorized by the Institute on the fire resistance of concrete building units. The tests were confined to hollow concrete block and building tile. The program was all laid out and the methods of making the block described in the report that was submitted last year and which appears printed in the *Proceedings*. I may remind the members that the occasion of making these tests was because of the hindrances, the difficulties, that the concrete products industry has found in getting an acceptance of their material by fire-insurance engineers and building commissioners, and the desire to know the real facts concerning the fire resistance of the concrete building unit, particularly in so far as they differed from solid concrete.

A. R. Small.—I was very glad, as a member of Committee P-5 to vote with Mr. Allen for this report which he has abstracted to you this evening, but I am not able to agree with every word of it. That is because since the report was written I have given a great deal of time to the study of the data we have secured at the Underwriters' Laboratories and have endeavored to put that into shape for our report and to defend what we hope are the proper conclusions of that report. I think all of us have had the experience of writing out or forming our conclusions first and then seeking to write a report to defend them, and that is substantially what we of the Laboratories' staff have done in this particular instance.

One of the points on which I am not able to agree with the report in the digest of conclusions, is that the influence of the five aggregate may be ignored. Mr. Richard L. Humphrey, in 1906 or 1907, directed a series of tests for the Geological Survey, in which were included some tests of block made exclusively of fine aggregate, and those blocks failed even more miserably than any of the blocks subjected to tests in this recent series. It so happens that in the samples used in our recent tests, with one single exception, the same fine aggregate was used throughout, and that was Fox River sand, and it has occurred to me that possibly the longitudinal cracking which has been so generally observed in all these hollow units in our tests, is due to that one very thing.

Mr. Allen has said in the report that the Structural Materials Laboratory has disclosed by its tests this this Fox River sand is a 45, 50 or 60 per cent silicious material. I think we are all willing to say that a highly silicious aggregate is not a desirable material when fire exposure performance is a consideration, and perhaps the 40 per cent of silicious material or silica that we find in the Fox River gravel or sand is the explanation after all of the web cracking in these units.

As to what is stability—we have attempted, in the Laboratories, to apply the standard specifications for fire tests of building material and

construction. Those tests require, as Mr. Allen has said, three fundamental performances, one being no passage of flame, another heat insulation, the critical point being when 300 deg. F. is reached on the unexposed face, and the third, for loaded assemblies, that they shall be stable. I do not know whether stability is required for a long time after the fire exposure or only during the period when exposure is occurring. A possible solution has occurred to some of those who have discussed this data; if we can find, by very careful consideration, and can compile, by plotting or otherwise, tables or data to show that longitudinal cracking does not occur in 100 per cent of the units and does not occur at any specific point in a given area, we will perhaps land eventually with the equivalent of uncracked units equal to the header courses in an 8-in. brick well; at least we will have the equivalent of headers in a brick wall and can consider that our entire wall area is substantially like that of a brick wall with headers every 6 or 7 courses. Where cracking has occurred, we have the equivalent of those portions of the area of a brick wall where the bricks are laid with the long dimension exposed; in other words, two courses of 4-in. brick. That may be a way of explaining to the satisfaction of engineers and city building inspectors, insurance engineers and rating authorities, that the stability of an 8-in. wall of cracked hollow units suffices. It will be up to such gentlemen more than to the staff of the Underwriters' Laboratories or any committee of the American Concrete Institute to determine just what stability means. This has been a very interesting work. A great many points have been brought out. The most important one, in view of the management of the Underwriter's Laboratories—a view shared heartily by its engineers, is that the use of these non-combustible units should be encouraged wherever possible. The continued use of combustible materials for the erection of buildings of the character of building in which this material is used is not a desirable thing from the point of view of public economy; and it is the function, I believe, of Underwriters' Laboratories to encourage the use of non-combustible materials in building construction, and certainly your products, these hollow concrete building units, classify as non-combustible and are to be preferred from that point of view. On that basis it will be my effort in drafting this report to have it advocate the use of these materials with proper limitations as to size and loading, wherever they are likely to be used as a substitute for combustible materials.

IRA H. WOOLSON.—The situation has been well stated both by Mr. Allen and Mr. Small. There was one thought that came to my mind as Mr. Small was speaking, and that was as to whether, in the analysis of the wall, they made careful enough study of the individual blocks to note to what extent the cracks were located on the face next to the fire, and what proportion may be located further back. That would have a bearing upon the question of bond still remaining in the wall after the fire. If ther were some webs left uncracked and others that were cracked midway of the wall or near the back, there would still be more or less of a bond-

ing action there that might resist sufficiently the eccentric load of a layer of floor beam ends carried on that wall. That is the problem that faces the insurance organizations; whether such a wall after a fire is sufficiently stable to support the eccentric load of the floor beams. I am told by insurance agents that they are frequently called upon to replace a concrete block wall after a fire because the inspector for the owner finds that a part of the blocks are cracked, and the owner says, "make the wall as good as it was originally." That means complete demolition, and a new wall. I think that is probably the reason why insurance companies have usually rated such walls in the same class as wooden construction. As far as they are concerned, the wall is a total loss. Now if we can show from these tests that there is still bond action enough in those walls to carry safely the eccentric load of the floor beams, possibly that objection can be overcome. I think it is a point that might well be given careful consideration, for, with Mr. Small, I feel that it is a type of construction that we should encourage and not discourage.

S. H. INGBERG (*by letter*).—In the reference made to the preliminary report on fire tests of brick walls by the Bureau of Standards the note relative to strength of brick masonry under fire conditions should not be interpreted as indicating any alarming condition. The probabilities are that all but 4-in. walls will satisfactorily sustain load throughout a 6-hour fire test. A series of fire tests under load are now in progress that will give definite information relative to the above.

The same applies to the effect of fire on masonry laid in lime mortar. While there is apparently a larger loss of strength of the mortar, the masonry does not lose strength in the same proportion since a high load can be sustained on the relatively thin beds. The present series of tests of walls under load is designed to give information on this particular also. All mortars lose strength on the exposed side of a wall subjected to fire, although the percentage loss is apparently higher with lime mortar than with cement-lime, or cement mortar, particularly if exposed before the lime has carbonated.

Recommended Practice for the Design and Construction of Concrete Dwelling Houses.

Progress Report of Committee S-5 on Reinforced-Concrete Houses.

I. FOREWORD.

In preparing this first draft of Recommended Practice for the Construction of Concrete Dwelling Houses, the Committee has recognized to a certain extent the natural limitations of loading and stresses in the dwelling house type. Methods of design applicable to large structures may, with safety, be simplified when applied to dwellings, and arbitrary regulation of wall thickness is justified. Certain facts of common knowledge in regard to the strength of concrete as compared with well-known types of lighter construction should be given expression in regulations for the heavier type.

No better excuse for submitting these regulations need be given than the comparative newness of the use of concrete for dwellings and the protection from fire loss gained thereby.

The report is not intended to be taken as complete at this time. It is so arranged, as may be seen by reference to the outline that additions may be conveniently made or revisions incorporated without disturbing the balance of the report.

The final chapter, which may be considered in the nature of an appendix, is to contain a program or requirements under which standards may be prepared for individual systems, units and forms of construction. It is proposed, with the co-operation of the proponents of each individual building material, system, unit and form of construction, that complete detailed standards may be prepared for each and promulgated so that they will serve as information for all who desire to use them. It is felt that this information will confer a needed service upon the building public and promote the proper and dependable use of each kind of dwelling house construction in a uniform and dependable manner and therefore the co-operation of the proponents of each individual kind of construction is earnestly requested.

In the main portion of the standards it is desired to set forth the general principles applicable to all kinds of construction and such general standards as may be applicable to concrete dwelling house construction without conflicting with any of the standards that may be subsequently promulgated. Here again suggestions are invited from everyone to the end that these chapters may be complete and fulfill their purpose.

II. MATERIALS.

(A) *Cement.*

Portland Cement.

1. Only standard portland cement that fulfills the specifications of the American Society for Testing Materials for portland cement shall be used in the construction of concrete and reinforced concrete dwelling houses.

(B) *Aggregates.*

General.

2. The use of aggregates in meeting the requirements of this Chapter is subject to the results of tests made at a testing laboratory of recognized standing, showing that the aggregates proposed for use will give a strength factor of safety of 5 on compressive specimens at 28 days. The tests shall be conducted in accordance with the Standard Methods of Making Compression Tests of Concrete of the American Society for Testing Materials.

(C) *Fine Aggregate.*

General Requirements.

3. Fine aggregate shall consist of sand, crushed stone or other inert materials having similar characteristics. It shall be free from such amounts of dust, lumps of clay, soft particles or other foreign substances likely to impair the strength or permanence of the mortar or of the concrete.

Grading.

4. Fine aggregate shall range in size from small to large, preferably within the following limits:

Passing a No. 50 Sieve: Not less than 30 per cent.
Passing a No. 4 Sieve: Not less than 95 per cent.

Impurities.

5. Fine aggregate consisting of natural sand shall not be used if it shows a darker color than the standard when tested in accordance with the Standard Method of Test for Organic Impurities in Sands for Concrete of the American Society for Testing Materials.

(D) *Coarse Aggregate.*

General Requirements.

6. Coarse aggregate shall consist of crushed rock or stone, gravel, crushed, air-cooled, weathered blast furnace slag or other inert material of similar character or combinations thereof having clean, hard, durable, strong uncoated particles free from injurious amounts of soft, friable, thin, or laminated pieces and from alkali, organic or other deleterious matter. Blast furnace slag for this purpose shall be free from flux-stone and should preferably weigh not less than 60 pounds per cubic foot and be seasoned in the bank for not less than one year.

Grading.

7. Coarse aggregate shall range in size from small to large, preferably within the following limits:

(a) For large sections or members:

Passing a No. 4 sieve: Not more than 5 per cent.
Passing a 1½-in. sieve: 95 per cent.
With no particles larger than 3 in.

(b) For sections or members less than four inches in the least dimension:

Passing a ½-in. sieve: 95 per cent.
With no particles larger than ¾ in.

8. Rubble aggregate shall consist of clean, hard, durable stone larger than coarse aggregate and not larger than one-man stone. **Rubble Aggregate.**

(E) *Water.*

9. Water for concrete shall be clean and free from oil, acid, alkali, organic matter or other deleterious substances. **General Requirements.**

(F) *Reinforcement.*

10. Metal reinforcement shall be of a quality and character meeting the requirements of the Standard Specifications for Billet Steel Concrete Reinforcement Bars of the American Society for Testing Materials. **General Requirements.**

11. Wire for concrete reinforcement shall conform to the requirements of the Tentative Specifications for Cold Drawn Steel Wire for Concrete Reinforcement of the American Society for Testing Materials. **Wire.**

12. The areas of deformed bars shall be determined by the minimum cross section thereof. **Deformed Bars.**

13. The quality of cast iron used in composite columns shall conform to the requirements of the Standard Specifications for Cast Iron Pipe and Special Castings of the American Society for Testing Materials. **Cast Iron.**

III. DESIGN.

(A) *Nomenclature.*

14. The symbols used in the following formulas are defined as follows:

A_c = gross sectional area of concrete.
A_s = the effective cross-sectional area of metal reinforcement in tension in beams or compression in columns.
b = width or rectangular beams or width of flange of T-Beam.
d = depth from compression surface of beam or slab to center of longitudinal tension reinforcement.
D = least horizontal dimension of column, shear or pilaster.
f_c = compressive unit stress in extreme fiber of concrete.
f_s = tensile unit stress in longitudinal reinforcement.
j = ratio of lever arm of resisting couple to depth "d."

L = span length of beam or slab, or unsupported length of column, pier or pilaster.

M = bending moment or moment of resistance in beam.

o = perimeter of the bar.

P = the carrying capacity of an axially loaded column.

p = ratio of effective area of tension reinforcement to effective area of concrete beams—A_s/bd.

u = the bond stress per square inch of superficial area of the bar.

V = maximum vertical shear, or shear at the critical section.

w = uniformity distributed load per unit length of beam or slab.

(B) *Working Unit Stresses.*

Concrete.

15. The following working unit stresses on concrete are based on a compressive strength of 2000 pounds per square inch at 28 days of the concrete cast in place:

Extreme fiber stress in flexure—
700 lb. per sq. in.

Extreme fiber stress compression, adjacent to support of continuous beams—
800 lb. per sq. in.

Shear in concrete—
40 lb. per sq. in.

Direct compression on plain concrete in piers, pedestals and footings—
500 lb. per sq. in.

Reinforcement.

16. Tensile stress on billet steel and rail steel bars—18,000 lb. per sq. in.

Masonry Walls.

17. The unit working stress for masonry walls, piers and pilasters built of concrete masonry units laid in portland cement mortar shall not exceed the following:

Hollow concrete block:	120 lb. per sq. in. of gross cross-sectional area.
Solid concrete block:	150 lb. per sq. in. of gross cross-sectional area.
Concrete brick:	250 lb. per sq. in. of gross cross-sectional area.
Cinder concrete block:	80 lb. per sq. in. of gross cross-sectional area.

(C) *Design Loads.*

General Requirements.

18. All parts of the structure shall be designed to carry the dead loads and in addition the live loads herein specified.

(a) Floors: Floor joists, slabs, arches and similar members—40 lb. per sq. ft.

(b) Roofs: Roof beams, rafters, purlins, etc.—30 lb. per sq. ft. of horizontal projection.

Where the climate is such that no snow loads occur, the roof live-loads may be taken as 20 lb. per sq. ft. of horizontal projection.

(D) *General Principles of Design.*

19. All parts of the structural supporting frame bearing loads or resisting stresses shall be designed so as to support the dead weight of the structure, including all permanent parts or members, and in addition all live-loads, without exceeding the allowable unit working stresses for the kind of materials, systems, units and forms of construction employed in the building. **General.**

20. All calculations and allowable unit working stresses shall, unless otherwise prescribed, conform to the standards of the Report of the Joint Committee on Concrete and Reinforced Concrete.

21. Whenever the structural parts or members are rigidly connected as to act as a unit in carrying loads and resisting stresses, the design of each part of the structural framework shall be computed according to the relative rigidities of the structural members and their connections.

22. Connections between the several parts of the structure shall be sufficiently strong and rigid to carry the loads imposed and to resist all lateral forces.

(E) *Beams.*

23. Beams shall not be considered as fixed, unless fixity is positively provided by monolithic construction with supports and by the use of appropriate reinforcement. **General.**

24. For beams continuous over two spans, the positive moment at center of span shall be: **Moment.**

$$M = \frac{wl^2}{10} \qquad (1)$$

and for negative moment over the middle support:

$$M = \frac{wl^2}{8} \qquad (2)$$

in which:

M = the bending moment.
w = load per lineal foot
l = the span face to face of supports

25. The resisting moment of the beam shall be computed by the following formulas:

$$\text{Resisting moment of steel — } M_s = f_s \times \tfrac{7}{8}d \times A_s \qquad (3)$$

$$\text{Resisting moment of concrete — } M_c = \frac{bd^2}{6} \times f_c \qquad (4)$$

M_s and M_c = Resisting moment of steel and concrete respectively.

Bond and Anchorage.

26. Bond stress on bars in beams shall be computed by the following formula:

$$u = \frac{V}{ojd} \tag{5}$$

27. u shall not exceed 80 lb. per sq. in. for plain bars, nor 100 lb. per sq. in. for deformed bars, unless anchorage of the bars is provided by anchoring the bar beyond the point of zero moment by means of U hooks having a radius of bend not less than four diameters of the bar and having a straight run of bar beyond the hook, of at least six bar diameters. If U hooks as above specified are used the bond stresses may be 160 lb. per sq. in. for plain bars and 200 lb. per sq. in. for deformed bars.

Diagonal Tension and Shear.

28. The shear v in reinforced-concrete beams shall be computed by the following formula:

$$v = \frac{V}{bjd} \tag{6}$$

where:

b = the breadth of the beam, and

v, V, b, j and d are the same as previously defined.

Shear reinforcement may be accomplished by

(a) Vertical stirrups or web reinforcing bars.

(b) Inclined stirrups or web reinforcing bars forming an angle a 30 deg. or more with the longitudinal bars.

(c) Longitudinal bars bent up.

29. Shear reinforcement need not be provided if the unit shear in the concrete does not exceed 40 lb. per sq. in. as computed by Formula (6).

30. Anchorage of all tension reinforcement shall be provided for as follows:

$$F = uox \tag{7}$$

where:

F = stress in bar

x = anchored length of bar.

31. The spacing a of web reinforcement shall be determined by the following formulas:

For values of a from 45 to 90°

$$s = \frac{A_v f_v}{b\ (v - .02f_c')\ \sin a} \tag{8}$$

and

$$s = \frac{A_v f_v\ (\sin a + \cos a)}{b\ (v - .02\ f_c')} \tag{9}$$

for values of a less than 45 deg.

32. Where both bent up bars and stirrups are used in conjunction for web reinforcement, the spacing s of points of bending up of longitudinal bars shall be determined by moment requirements. The unit shear value v_1 of these bars shall be determined by the formula:

For values of α from 45 deg. to 90 deg.

$$v_1 = \frac{A_v f_v}{bs \sin \alpha} \qquad (10)$$

and for values of α less than 45 deg.

$$v_1 = \frac{A_v f_v (\sin \alpha - \cos \alpha)}{bs} \qquad (11)$$

33. The spacing of stirrups to take the remaining shear not provided for by the concrete and bent up longitudinal bars shall be determined from Formulas (8) and (9).

A_v = area of bent up bars in any one plane

and

f_v = unit working stress of steel.

(F) *Flat Slabs.*

34. Flat slab floors shall be designed by competent engineers according to the methods for flat slab design provided in the recommendations of the Joint Committee on Concrete and Reinforced-Concrete. In the vast majority of cases the girder and slab beam and slab type of floor will prove most economical because of the light loads on dwelling house floors. **Flat Slabs.**

(G) *Columns.*

35. Reinforced-concrete columns shall be of the type known as tied columns and shall not have a ratio of length to diameter or least cross sectional dimension less than 25. The longitudinal reinforcement shall be not less than one-half of 1 per cent nor more than 3 per cent of the column. The lateral type shall be not less than ¼ inch in diameter and spaced not more than 8 inches apart. **General.**

36. The carrying capacity of tied columns shall be determined by the following formula: **Formula.**

$$P = 400 \ (A_c + n \ A_s \qquad (12)$$

in which

P = the carrying capacity of an axially loaded column
A_c = net area of the concrete
A_s = cross-sectional area of longitudinal steel
n = ratio of modulus of elasticity of steel to that of concrete.

(H) *Piers and Pilasters.*

Reduction for Height.

37. Where the height of piers and pilasters between lateral supports exceeds five times the least lateral dimension, the unit working stresses shall be reduced by the following formula:

$$f' = f \left(1.25 \text{ minus } \frac{L}{20\,D} \right) \qquad (13)$$

where

f' = the reduced allowable unit stress in pounds per square inch.

(I) *Walls.*

General.

38. Monolithic concrete construction containing not more than 0.2 per cent of reinforcement so placed as to reinforce the structure against stresses resulting from temperature changes shall be classed as plain concrete construction.

39. All parts of monolithic supporting walls shall be reinforced at floor and roof connections and at places subject to concentrated loads, so as to resist lateral forces and to distribute the loads to all parts of the supporting walls.

Solid Bearing Walls, Cast in Place.

40. Solid bearing walls of cast-in-place concrete need not be reinforced except as herein required.

41. Where reinforcement is provided to assist in carrying loads and resisting stresses, the thickness of bearing walls of cast-in-place concrete construction may be less than as set forth for bearing walls, provided they shall be designed and constructed so as to carry all loads and resist all forces without exceeding the unit working stresses prescribed by the Report of the Joint Committee or prescribed by this standard as the conditions require.

Precast Unit Construction.

42. Precast units for construction of concrete dwelling houses shall be reinforced where necessary to prevent breakage in handling and erection.

43. All detail connections shall be of sufficient size and adequately reinforced to transmit all vertical and lateral forces.

44. Reinforcement in each unit shall be extended in such a manner that it may be hooked, looped or otherwise rigidly fastened to the reinforcement in contiguous units, in such a manner that the resulting structure will carry loads and resist stresses as a unit.

Structural Frame with Curtain Walls.

45. Dwelling houses constructed with structural framework supporting floors, roofs and enclosing walls shall be designed in accordance with accepted principles of engineering analysis and shall be of sufficient strength and rigidity to carry all loads and resist all lateral forces.

46. Design computations and methods of construction not provided for herein, shall conform to the requirements of the Report of the Joint Committee on Concrete and Reinforced Concrete.

47. The number of stories in dwelling houses having concrete masonry walls of the minimum thicknesses herein permitted is limited only by the permissible height of the walls. **Number of Stories.**

48. Unless otherwise provided herein, the height of any wall built of concrete masonry shall not exceed twenty times the thickness of such walls unless adequately braced by floors or roofs or by masonry piers or pilasters or buttresses or masonry cross-walls. **Height of Walls.**

49. The height of walls as herein regulated shall be measured from the top of the foundation wall or from a supporting girder or from the foundation supports or other immediate support to the top of the wall under consideration.

50. Foundation walls built of concrete masonry units shall be at least as thick as the walls, piers, pilasters and buttresses that they support, except that walls required to be in excess of eight inches in thickness may be not less than eight inches in thickness above the slope of basement stairs for a distance of not to exceed ten feet horizontal provided the walls above are properly supported. **Thickness of Foundation Walls**

51. Where concrete masonry foundation walls serve as cellar or basement enclosing walls supporting not to exceed five feet of earth pressures, the minimum allowable thickness shall be eight inches. Where they serve as cellar or basement enclosing walls supporting earth pressures in excess of five feet in height, the thickness of these walls shall be increased as required for the height of the slope and the nature of the earth supported.

52. Where concrete masonry foundation walls serve as cellar or basement walls, (enclosing or otherwise) and do not resist earth pressures, they shall be at least as thick as the walls they support provided the clear story height of the cellar or basement does not exceed 9 ft. Where the clear story height of the cellar or basement exceeds 9 ft., these walls shall be at least 2 in. thicker than the walls, piers, pilasters or buttresses they support.

53. For walls more than 35 ft. in height, the minimum allowable thickness shall be 8 in. for the uppermost 14 ft. of the wall when reinforced by buttresses, pilasters, or masonry cross walls, or stud cross walls securely anchored, all of which reinforcements shall be not more than twenty feet apart, and when not so reinforced, the minimum allowable thickness shall be twelve inches, and shall be: **Thickness of Walls Above Foundations.**

12 in. thick for the next lower 28 ft. of the wall,
16 in. thick for the next lower 36 ft. of the wall.

54. For walls not over 35 ft. in height, the minimum allowable thickness shall be 8 in. for the uppermost 25 ft. in height of the wall when reinforced by pilasters, buttresses, or masonry cross walls, or stud cross walls securely anchored, all of which reinforcements shall be not more than 25 ft. apart, and when not so reinforced the minimum allowable thickness shall be twelve inches, and shall be:

12 in. thick for the next lower 15 ft. of the wall.

55. The minimum allowable thickness of the walls may be eight inches for the full height of the wall if not exceeding a height of 25 ft. to the square of the roof, or 35 ft. to the peak of the gable, provided they are reinforced by buttresses, pilasters, masonry cross walls or stud cross walls securely anchored not farther apart than 25 ft. Buildings having mansard roofs are not to be taken as coming within the regulations of this paragraph.

56. When the foregoing reinforcements are spaced closer together or farther apart than 25 ft., the maximum allowable height to which the walls may be built shall be computed by the use of the following formula:

$$h^1 \text{ equals } \frac{h^2}{L} \qquad (14)$$

where:

h^1 equals the maximum allowable height of the wall reinforced as required
h equals the height given in the foregoing
L equals the distance apart of the required reinforcements.

57. Solid bearing walls of cast-in-place concrete may be two inches less in thickness than permitted for walls built of masonry units, but shall be not less than six inches in thickness.

58. Hollow bearing walls of cast-in-place concrete shall have a total thickness of materials of the inner and outer walls at least as great as herein required for solid bearing walls.

59. All changes in thicknesses of masonry walls may occur at the level of the floor nearest to the height specified except that on stair runs they may be carried to the underside of the stair flight above the height at which the thickness is required to change.

IV. CONSTRUCTION.

General. 60. The quality of concrete masonry units, including the materials of which they are made, the strength and weather resistance, and such other matters as describe the construction of the masonry units shall be such as will conform to the requirements of Committee P-1, of the American Concrete Institute.

61. Hollow bearing walls of cast-in-place concrete construction shall have the inner and outer walls securely braced together by non-corroding wires spaced not farther apart than twelve inches vertically and fifteen inches horizontally and properly embedded in the concrete and hooked at the ends. Other methods of bracing may be used provided they are sufficiently strong to bring the walls into common action. **Hollow Bearing Walls Cast in Place.**

62. Reinforcement shall be placed in both the inner and outer walls not less than No. 9 wire spaced not to exceed twelve inches vertically. Such reinforcement shall be placed at the mid-height of each succeeding horizontal course. Where the horizontal courses exceed twelve inches in height the amount of reinforcement in each horizontal course shall be at least as great as here required per foot of height of walls.

63. Reinforcement shall also be placed in the walls over all openings sufficient to carry all loads and prevent cracking but not less than two-tenths of one per cent based upon one foot in vertical height of the wall.

64. All concrete masonry units shall be laid in portland cement mortar consisting of one part portland cement, to not more than three parts by measure of fine aggregate. **Mortar]**

65. All masonry units shall be laid with the vertical joints broken, and with all courses thoroughly bonded in the proper manner for the kind of unit used. All masonry facing or backing made of concrete masonry units shall be bonded to the masonry backing or facing of other masonry units with masonry bond in the proper manner for the kind of masonry units used, (the use of wall ties shall not be considered as sufficient bonding where the masonry facing is to be counted in the thickness of the wall, pier or pilaster). **Joints and Bond**

66. Whenever the concrete masonry units are to be covered with stucco or plaster, the surfaces to receive the stucco or plaster shall be treated so as to insure the dependable attachment of the stucco or plaster. **Stucco.**

67. All bearings on concrete masonry units shall be not less than four inches. Where vertical cell construction is used, the load shall be distributed by means of metal or masonry bearing plates of sufficient thickness to distribute the imposed load or the supporting course shall be filled with concrete, or other equivalent method of construction shall be used. **Bearing on Walls**

68. Wherever a change occurs in the thickness of walls, piers and pilasters made of concrete masonry units, the units shall be laid with a sufficient masonry bond at the top of the thicker section, and for hollow units laid with the cells vertically unless the longitudinal and cross walls of the units are properly superimposed the bearing loads shall be distributed upon the wall below by means of masonry or metal bearing loads, or the supporting course made of solid units, or solidly filled with concrete, or other equivalent method of construction may be used.

69. No wood shall be built into or made a part of any bearing wall made of concrete masonry units except for joist or beam bearing blocks or nailing strips not over 1 in. in width by the thickness of the mortar joint, provided, however, that wood bearing plates may be built into such walls for the purpose of supporting the roof rafters or ceiling joists when the distance from the same to the top of the masonry walls does not exceed 3 feet.

Waterproofing. 70. All concrete masonry foundation walls of hollow units shall be thoroughly waterproofed on the outside or else made of waterproofed units and laid in waterproof mortar, or else the excess ground water shall be carried away from the foundation walls by drains.

DISCUSSION.

S. C. HOLLISTER.—There are certain phases of this report to which I should like to call your attention. One is, that in Section 10, specification is made for the use of billet steel concrete reinforcement bars. In Section 16 the tensile stress in reinforcement bars covers both billet steel bars and rail steel bars, rail steel bars not having been provided for in the specification for reinforcement.

The specification for cement, it might be noted, is a specification of the American Concrete Institute as well as the American Society for Testing Materials, and could be referred to as such.

The whole section, beginning with paragraph 23 and extending through paragraph 25, covering the design of beams, is entirely inadequate. In the first place, coefficients for bending moment in beams cover only one type of beam, that for a beam continuous for two spans; it does not provide for beams of one span or more than two spans, or for different conditions of restraint. In Section 25 there is no provision made for design of a T-beam which should be provided for in a reinforced concrete floor. The formulas 3 and 4, especially formula 3 in Section 25, are not adequate for a standard.

In paragraph 34 under flat slabs is the statement that flat slab floors shall be designed by competent engineers, etc. That is decidedly not a specification.

The section on walls, beginning with paragraph 38, refers to the matter of temperature reinforcement of two-tenths of 1 per cent in wall construction. I think this whole scheme of temperature reinforcement has long since been exploded.

There is a decided lack of clarity in the report concerning the matter of concrete masonry, as to what constitutes concrete masonry, if it is differentiated from cast-in-place concrete or concrete pre-cast units.

W. A. SLATER.—What section was that?

MR. HOLLISTER.—That relates to the whole report, but it especially relates to paragraph 60, for example.

I should like to call general attention to the fact that there are many committees of the Institute who are preparing basic specifications for various types of units and various conditions of design. This includes also the tentative standard of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete. In a specification of this kind which undertakes to group together a number of these basic specifications, it would seem to me that it would be far preferable to refer to these basic specifications than to undertake to duplicate them in part. It is exceedingly difficult to extract from this specification and that specification only a portion of the basic specification without leaving out very essential elements. One way to do would be to leave out the basic specification entirely and instead refer to it; as, for instance, in the case of the

specification for cement under the heading "Cement." There are already several paragraphs throughout the report which refer to such other basic specifications as that of the Joint Committee. It would seem that this scheme might be followed even more generally than it is followed in the present presentation of the report.

I wish to call attention also to the section on piers and pilasters, paragraph 37. This is a very unusual statement for the design of a pier or pilaster. Is should preferably be supported by some data as to what this formula was based upon. There is nothing to indicate in the report what consideration was given in order to arrive at formula 13.

MR. FERGUSON.—May I call attention to the fact that paragraph 37, standard masonry formulas for determining the allowable stress on masonry piers, was taken from standard textbooks. If the books are wrong, we will be glad to correct the formulas. I think a great many of Mr. Hollister's suggestions might well be adopted, but I would like to leave this with the Institute. In connection with too much repetition of reference to other standards—we do not want this report to be just one continuous reference to this standard, that standard and the other; let the man who is reading not have to go searching to find what part of that standard pertains particularly to concrete houses.

A. C. IRWIN.—Mr. Hollister's remarks contain many things of merit. I am sure, however, that Mr. Hollister has not considered the things that the committee has considered in preparing its report. In the first place, we are preparing a report for concrete houses and not for a large office building or a big warehouse. There is a distinct difference, which we have pointed out in our foreword. That explains a good many of the things to which Mr. Hollister takes exception. The basic specification mentioned by him as being those specifications already adopted by the Institute as standard, do not necessarily apply to this report for the reason just stated, that this is a report for concrete houses. A small unit which is recognized by the building code committee of the Department of Commerce and by the National Board of Fire Underwriters as being a type of structure that requires separate consideration in order that the houses evolved shall be economical and not be loaded down with a lot of unnecessary requirements. Therefore we are not in a position to copy exactly the standard specifications, even in regard to material; as, for instance, aggregates used in concrete, in the construction of a concrete house, do not need to come up to the rigid specifications required for an important building, so that we cannot copy these specifications completely, and to cut in here and there and refer to this serial number and that report of such and such committee would make our report a disjointed, disconnected, unusable proposition.

Now with respect to some of the details; Mr. Hollister went so fast that it bothered me a good deal to keep up with him. Maybe I will miss some of them. In regard to the formulas for the design of beams—I will admit that probably we ought to put in a formula for the design of a

simple beam, and we are obliged to Mr. Hollister for that suggestion. I am not, however, willing to admit that we need to include a detailed account of the methods of design of a T-beam in building a house which is to be subjected to 30 lb. per ft. live-load with spans not exceeding 15 or 16 ft. The attempt at refinement in a case of that kind will not be worth the time, and the specifications should be such that they can be used by a man who is not specially versed in the method of reinforced-concrete design, and the simpler and shorter we can make them the better. And that is the principle we have gone on and that is the reason. It was not forgotten; that is the reason why we omitted formulas for the design of T-beams.

In regard to flat-slab construction—flat-slab construction is distinctly a type of construction applicable to heavy warehouses and not to a small dwelling house at all; so that instead of passing the thing up entirely, we simply put in this paragraph which recites that in case the unusual, I would be almost willing to go so far as to say the never-to-be case in which a man will want to build a flat-slab design in an ordinary dwelling house—we simply point out the fact that this is a type of construction of sufficient intricacy as to require an experienced designer to evolve it.

In regard to temperature reinforcement, par. 38, I call attention to the fact that that distinction as between plain concrete and reinforced-concrete which we have again repeated, has occurred in no less than four specifications that I remember at the present time. It is not anything new. It almost by common consent has been agreed upon, tacitly at least as being a distinction as between plain and reinforced-concrete. I will be interested to know, for my personal information when the use of reinforcement for temperature purposes has been exploded. That is brand new news to me.

In regard to construction, par. 60, Concrete Masonry Units—I cannot think of any more definite way to define them than by calling them concrete masonry units—a certain number of publications have already been put out in which these units are described as concrete masonry units. If the convention thinks there is some better term, or if Mr. Hollister can suggest a better term to use, I am sure the committee would be very glad to have it.

MR. HOLLISTER.—Replying first to Mr. Irwin's general thought on the subject—It is true that there are conditions which surround the design of a concrete house that are quite different from the conditions surrounding the design of a larger building. It was not my purpose, at least, to eliminate the consideration of such difference. On the other hand, when you are designing a concrete house, there are certain things, which, as a matter of good engineering principle, and as a matter of good understanding of the use of concrete, should be considered. Take, for example, the design of a T-beam: If you have a floor slab supported directly upon a beam cast with it, it is a T-beam,—it is not a rectangular beam and cannot be designed as a rectangular beam. It is not uncommon

to use spans of 18 and 20 ft. in residential construction. There is no limit to the size of house under these specifications, and an 18- or 20-ft. span is not unlikely. An 18- or 20-foot span in T-beam construction requires design, and design of T-beam should not, in my opinion, be omitted.

With reference to the section on flat slabs, I call attention specifically to the expression "flat slab floors shall be designed by competent engineers." I consider that not a good form for specifications.

Under Section 38 is covered the matter of temperature changes and reinforcing against them. If you imbed a steel bar in a concrete member and submit the combined unit to a change of temperature, the steel and concrete expand very nearly at the same rate; there is some difference in certain mixes in the co-efficient of thermal expansion of the concrete from the co-efficient of thermal expansion of the steel. Taking the maximum difference of such expansions in a building 200 ft. long, for instance, an expansion of 7½ ft. would be necessary to develop 16,000 lb. per sq. in. in any reinforcement for temperature; that, of course, would require an enormous temperature to develop any such thing, and any ordinary temperature will develop such a small stress in the reinforcement, that is relatively negligible. It is true, as Mr. Irwin states, that there are a number of specifications that continue to use expressions such as the inclusion of two-tenths of 1 per cent reinforcement for temperature changes. That may be in the specification, but that is not the way that "reinforcement" acts.

Under Section 60, the reference is made to concrete masonry. This occurs also in Section 51 and 52, which begin, "concrete masonry foundation walls." It is not clear just what is meant by "concrete masonry walls." It is a question of definition which should be included in the report in order to relieve the ambiguity.

MR. FERGUSON.—Plenty of Mr. Hollister's suggestions are good. Of course we have not attempted to get this work complete. It had to be prepared since the first of December because the committee did not get to work before that time, owing to the fact that its membership was not certain. However, it is perfectly true that many of our proposals will have to be scaled down to the question of certain houses as against the intricate designs necessary for larger or more voluminous structures. The question of T-beam design might well be introduced, and the committee will consider those points. The question of design of concrete masonry was left out because of the fact that we were afraid if we defined it too much in detail, that would be objected to. We will be very glad to ponder over again the question of putting that in. I have written a number of building codes myself in which I purposely and by experience left that definition clear out because of the fact that when it was in it limited the activities of concrete products in masonry.

REPORT OF COMMITTEE E-8, ON EXPANSION JOINTS IN CONCRETE CONSTRUCTION.

The Committee on Expansion Joints in Concrete Construction submits, for the purpose of discussion, the following summary of the problem of expansion joints, with a certain amount of information concerning the materials usually employed in such work. Typical sections are shown of expansion joints of different types.

The Problem.—Concrete cracks—sometimes it doesn't.

Why does it crack?—why doesn't it crack?

Does moisture, shrinkage, temperature, freedom to move (unrestrained, partially restained or restrained) steel reinforcing, expansion joints, shape, size, covering and numerous other factors have anything to do with the presence or absence of cracks? If so, what are their relative effects?

Sometimes it is essential to avoid all cracks.

Oftentimes it is desirable to avoid all cracks.

Usually it is desirable to avoid all large and unsightly cracks.

Occasionally it is permissible to allow it to crack or not crack, as it pleases.

Always it is essential to avoid the disastrous effects of buckling.

Expansion joints are expensive, bothersome and usually unsightly. Can they be eliminated?

If they cannot be eliminated how often should they be put in and what is the best type of expansion joint to use in each case?

Force Exerted by Concrete as Temperature Changes.—A concrete stick of 1 sq. in. cross-sectional area (regardless of its length) requires the application of a force of 20 lb. to keep it from elongating under a rise of temperature of 1 deg. Fahrenheit. Hence a 10 x 20-in. concrete strut subjected to a rise in temperature of 60 deg. will push against a restraint (completely preventing its elongation) with a force equal to 20 lb. $\times$ 10 in. $\times$ 20 in. $\times$ 60 deg. = 240,000 lb.

The above figures are based on a temperature coefficient for concrete of 0.000,006,5 a coefficient of elasticity of a little over 3,000,000 and the formula, Pressure equals Coeff. of E. times Temp. Coeff. times change in temperature in degrees Fahrenheit, ($P \times E \times .000{,}006{,}5 \times 60$). Please note that the concrete strut does not contain any reinforcing steel. If it

contained 1 per cent reinforcing steel the push will be 261,600 lb. (assuming E for steel to be about 30,000,000) as per these figures:

$$P = 30{,}000{,}000 \times .000{,}006{,}5 \times 60 \text{ deg.} \times 1 \text{ per cent of } (10 \times 20 \text{ in.}) = 24{,}000 \text{ lb.}$$

$$P = 3{,}000{,}000 \times .000{,}006{,}5 \times 60 \text{ deg.} \times 99 \text{ per cent of } (10 \times 20 \text{ in.}) = 237{,}600 \text{ lb.}$$

$$261{,}600 \text{ lb.}$$

Expansion Joints in Concrete.—Fundamentally the problem of designing an expansion joint is that of finding a substance or an arrangement of parts that always will keep just filled the gap between the moving concrete parts. In addition structural strength, resistance to displacement, tightness and durability are often demanded.

It is conceivable that a rubber-like substance would do. Perhaps a sponge-like compressible flax-straw board, coated with asphalt and in which small stones, (bonding dowel-fashion in the asphalt and the green concrete) are embedded would do. Perhaps some cellulose compound could be made to serve.

The opportunity for invention and discovery to find a substance that would do is wide open. So far very little has been accomplished. An arrangement of parts, usually sheet metal and a filler, is commonly used. Sketches of such, classified according to the conditions they must combat are shown elsewhere.

The sheet metals most frequently used are copper and lead. A comparison of the properties of these two metals seems to indicate that in most cases copper is the more suitable.

Copper:

Reddish color

Highly malleable and ductile

Density varies slightly average value 8.9

Weight per cubic ft. 556 lbs.

Melts at 1981° Fahr.

When exposed to ordinary air becomes oxidized turning to a black color, but the coating is protective and the oxidizing process is not progressive as with iron and steel.

When exposed to moist air containing CO_2 it becomes coated with green basic carbonate. Is also affected by SO_2. It resists the action of hydrochloric, sulphuric and strong nitric acids at ordinary temperatures but is acted upon by dilute nitric acid.

Cold rolling and drawing greatly increase its tensile properties but reduces its electrical conductivity by from 2 to 4 per cent.

Commercial electrolytic copper as commonly gotten in the market is about 99.9 per cent chemically pure copper. The poorest grades of copper, such as casting copper, run about 99 per cent chemically pure copper.

Temperature coefficient of expansion per degree Fahrenheit is .000,009,22.

Annealed copper wire has a tensile strength of about 34,000 lb. per sq. in.

Cold drawing copper to the medium hard stage gives it an ultimate tensile strength of about 45,000 lb. per sq. in., to the hard stage about 55,000 lb. per sq. in.

Cast copper ranges in ultimate tensile strength from 20,000 to 30,000 lb. per sq. in. and has a crushing strength of about 40,000 lb. per sq. in.

The fracture of forged, rolled or drawn copper is fibrous, with a silky lustre.

An excess of the suboxid of copper Cu_2O causes copper to be brittle.

Annealed copper has no very definite elastic limit and in fact commences to set (flow under stress) at comparatively low stresses. It also has no yield point. Copper of medium or of full hardness behaves differently and exhibits a fairly definite elastic limit but no yield point. However in the strictest sense of the term, copper probably has no absolutely definite elastic limit whatever.

A conservative value for the elastic limit for copper of all degrees of hardness is about 50 per cent of the ultimate.

The coefficient of elasticity is about 12,000,000 for annealed wire—16,000,000 for hard drawn wire.

Specific heat coefficient ranges from 0.0862 to 0.0928.

Thermal conductivity at ordinary temperatures in g-cal (cm cube) per sec. per deg. cent. is about .72.

The thickness of cold rolled copper sheets obtainable on the market are as per the American (Brown and Sharpe's) gauge.

Rolled or forged copper has an ultimate strength of 30,000 lb. per sq. in. between 60 deg. and 300 deg. F. The strength decreases rapidly with increase of temperature. Electrolytic copper drawn into wire has an ultimate strength of 47,000 lb. per sq. in.

Modulus of elasticity of rolled copper = 16,000,000

Ultimate strength in tension of rolled copper = 28,500 to 33,000

Modulus of elasticity of copper castings = 12,000,000

Ultimate strength in tension of copper castings
= 22,000 lb. per sq. in.

Elastic strength in tension of copper castings
= 6000 lb. per sq. in.

Ordinary working fiber stress of rolled copper in tension =
8500 for dead-loads
4300 for live-loads

The average price of sheet copper is about 25 ct. per pound.

Bond of Copper to Concrete.—See Waterworks Handbook, Flinn, P. 270, 1916, Edition. Tests with 18 oz. copper inside a standard briquette 1:2 mix 28 days old gave the following results:

Type	Pull to Break concrete	Pull to bond between copper and concrete
Without Copper	240 lb.	......
Cleaned Copper	308 "	364 lb.
Uncleaned Copper	346 "	431 "

About 2 sq. in. of copper was in contact; adhesion per sq. in. was 215 lb. with uncleaned copper and 182 lb. with cleaned (Eng. Rec. March 2, 1912).

Lead.

Density is 11.3

Weight per cu. ft. 706 lb.

Coefficient of expansion per degree Fahrenheit = .0000155

Lead melts at 625 deg. Fahrenheit

Thermal conductivity (g-cal per cm cube per deg. cent. per sec.) = .084

Temperature coefficient = .00387

Ultimate tensile strength of rolled lead = 2700 lb. per sq. in.

Elastic limit tensile strength of rolled lead = 800 lb. per sq. in.

Lead dissolves in water to a slight extent, but carbonates or sulfates of lime deposited upon it from the water form a film on the surface which prevents further action.

Lead has a low strength, is almost devoid of elasticity, is very plastic so that it flows readily under stress.

The weights per sq. ft. of sheet lead ordinarily rolled are:

Weight, lb.—2½, 3, 3½, 4, 4½, 5, 6, 7, 8, 9, 10.

Thickness, inches—1/24, 1/20, 1/18, 1/16, 1/14, 1/12, 1/10, 1/9, 1/7, 9/64, 1/6 and upward.

Stock rolls average 20 ft. in length.

The average price of sheet lead is about 12 ct. per pound.

Several cases of damaging corrosion of lead pipe from contact with mortar and cement are known.

In ordinary atmospheric corrosion lead is one of the most durable of the common metals, undergoing no change in dry air or in water perfectly free from air; it is only slightly affected by hard waters or dilute solutions of either hydrochloric or sulphuric acids; but is readily dissolved by water high in nitrates and by dilute nitric acid; waters which actively corrode lead are those with a slightly acid reaction, from peaty swamps; soft waters are particularly unsuited for conveyance in lead pipes. Lead water pipes should be kept full of water all the time to prevent deterioration. See Page 617, of Waterworks Hankbook by Flinn, Weston and Bogert.

Filler.

Poured.

Asphalt.

Rubber.

Preformed.

Elastite.

Philip Carey Co., Cincinnati, Ohio.

Serviciscd Products,

Servicised Products Co.,
1st National Bank Bldg.
Chicago, Ill.

Aztec Expansion Joint,

The United States Asphalt Refining Co.,
1108 Conway Bldg.,
Chicago, Ill.

Minwax,

Minwax Company, Inc.,
327 So. La Salle Street,
Chicago, Ill.

CLASSIFICATION OF EXPANSION JOINTS.

—9 Classes, 33 Groups—

		Class Designation, where used.
Group A.	Air Pressure, One Side.	APOS—Air Pressure Pipes.
	Liquid Pressure, One Side.	LPOS—Exterior Tank Walls, Tunnel Walls.
	Granular Pressure, One Side.	GPOS—Exterior Sand and Grain Bin Walls, Tunnel Walls.
Group B.	Air Pressure, Two Sides.	APTS—Interior Walls of a Multiple Air Main.
	Liquid Pressure, Two Sides.	LPTS—Interior Walls of a System of Water Tanks.
	Granular Pressure, Two Sides.	GPTS—Interior Walls of Multiple Sand or Grain Bins.
Group C.	No Pressure, Two Sides.	NPTS—Viaduct Railings, Parapet Walls.
	No Pressure, Two Sides, Rain Tight.	NPTSRT—Over Sub-sidewalk Spaces, Subways, Viaducts.
	No Pressure, Two Sides, Wind Tight.	NPTSWT—In Buildings.

The class designation partially sets forth in an abbreviated form the conditions the joint must combat.

These nine classes of expansion joints form three groups of three classes each.

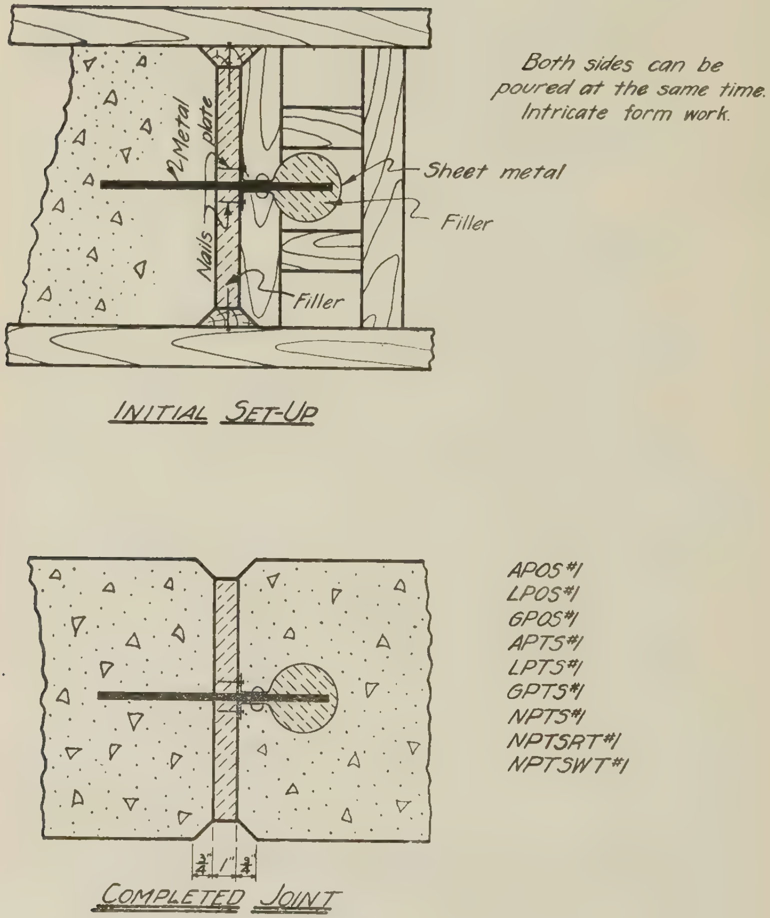

Both sides can be poured at the same time. Intricate form work.
½ Metal plate
Sheet metal
Filler
Nails
Filler
INITIAL SET-UP
APOS#1
LPOS#1
GPOS#1
APTS#1
LPTS#1
GPTS#1
NPTS#1
NPTSRT#1
NPTSWT#1
¾"
1"
¾"
COMPLETED JOINT

FIG. 1.

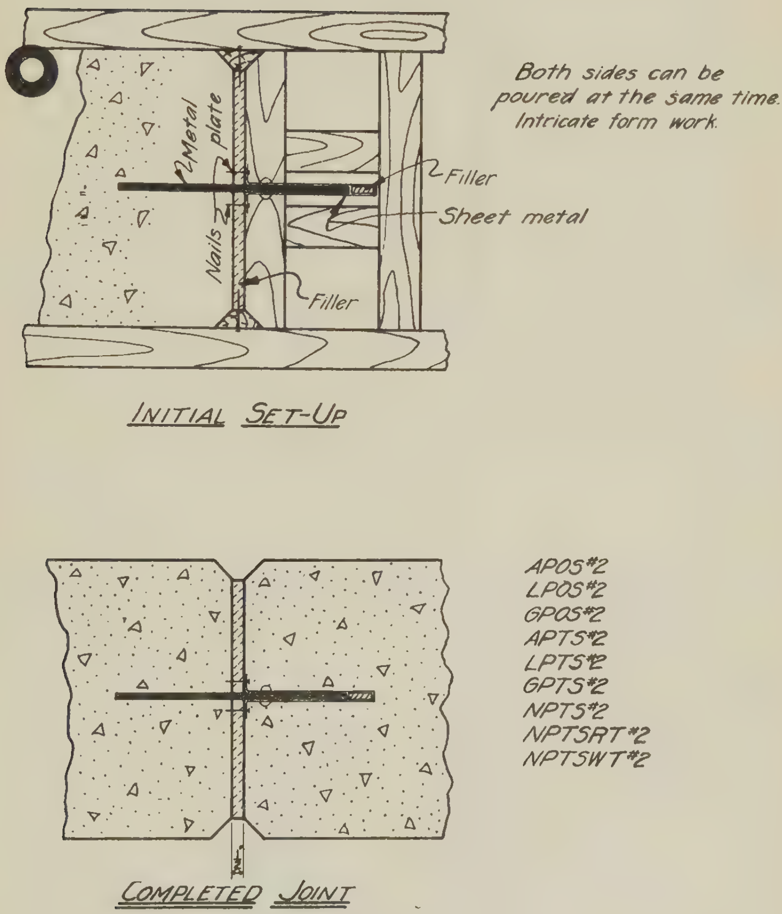

Both sides can be
poured at the same time.
Intricate form work.
Metal plate
Nails
Filler
Sheet metal
Filler
INITIAL SET-UP
APOS#2
LPOS#2
GPOS#2
APTS#2
LPTS#2
GPTS#2
NPTS#2
NPTSRT#2
NPTSWT#2
COMPLETED JOINT

FIG. 2

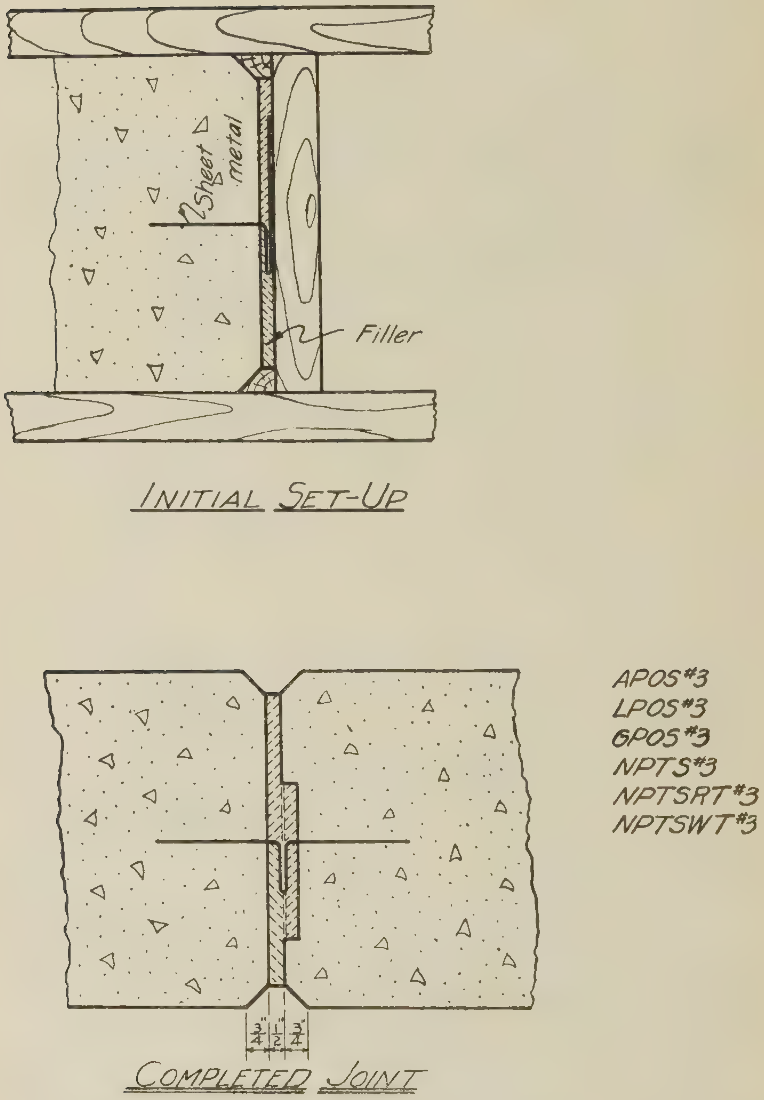

Sheet metal
Filler
INITIAL SET-UP
APOS#3
LPOS#3
6POS#3
NPTS#3
NPTSRT#3
NPTSWT#3
3/4"
1/2"
3/4"
COMPLETED JOINT

FIG. 3.

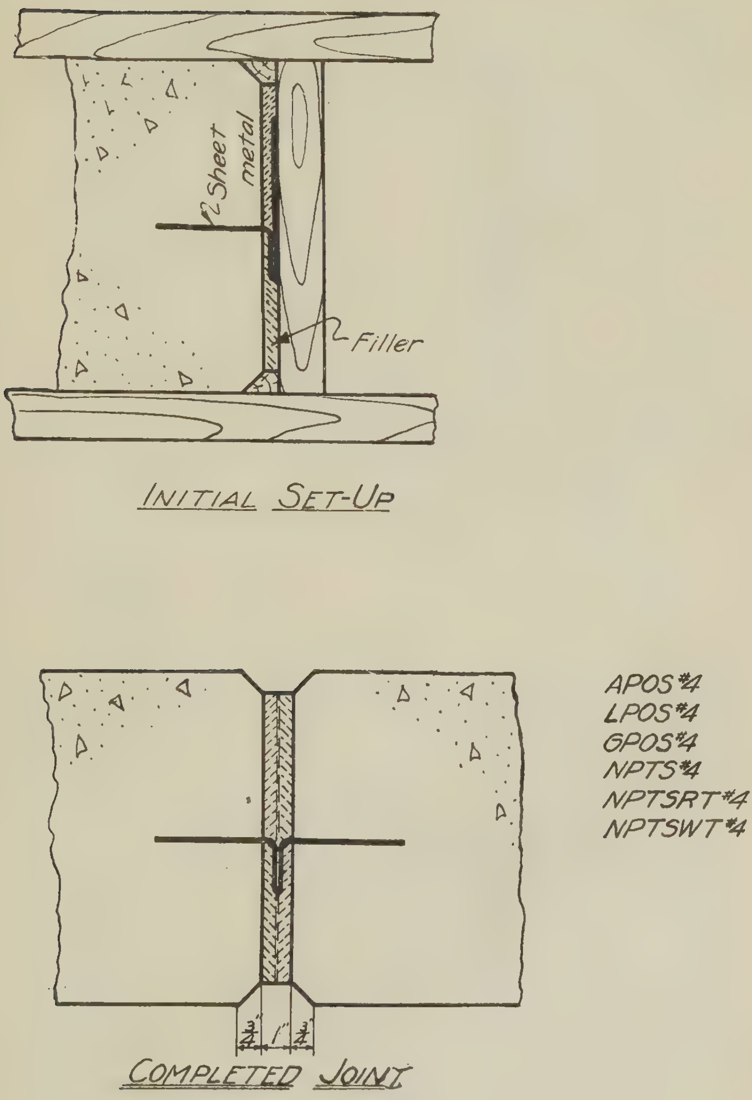

Sheet metal
Filler
INITIAL SET-UP
APOS#4
LPOS#4
GPOS#4
NPTS#4
NPTSRT#4
NPTSWT#4
3/4"
1"
3/4"
COMPLETED JOINT

FIG. 4.

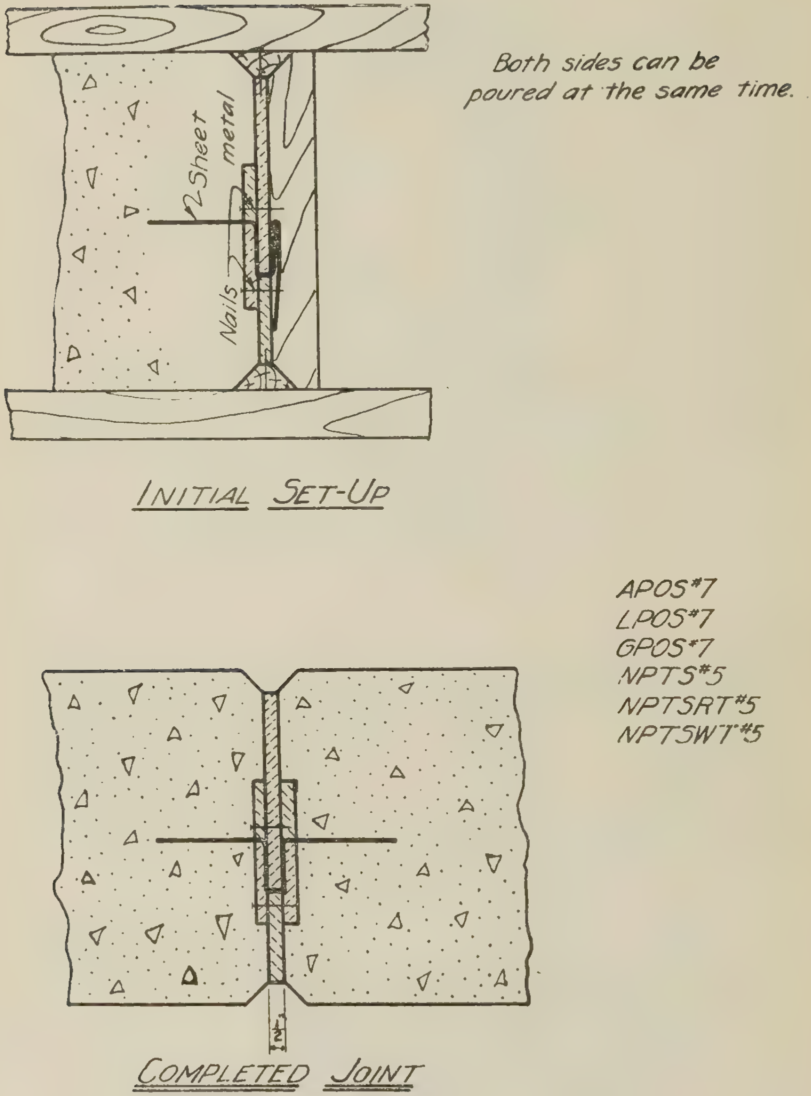

Both sides can be poured at the same time.
2 Sheet metal
Nails
INITIAL SET-UP
APOS*7
LPOS*7
GPOS*7
NPTS*5
NPTSRT*5
NPTSWT*5
COMPLETED JOINT

FIG. 5.

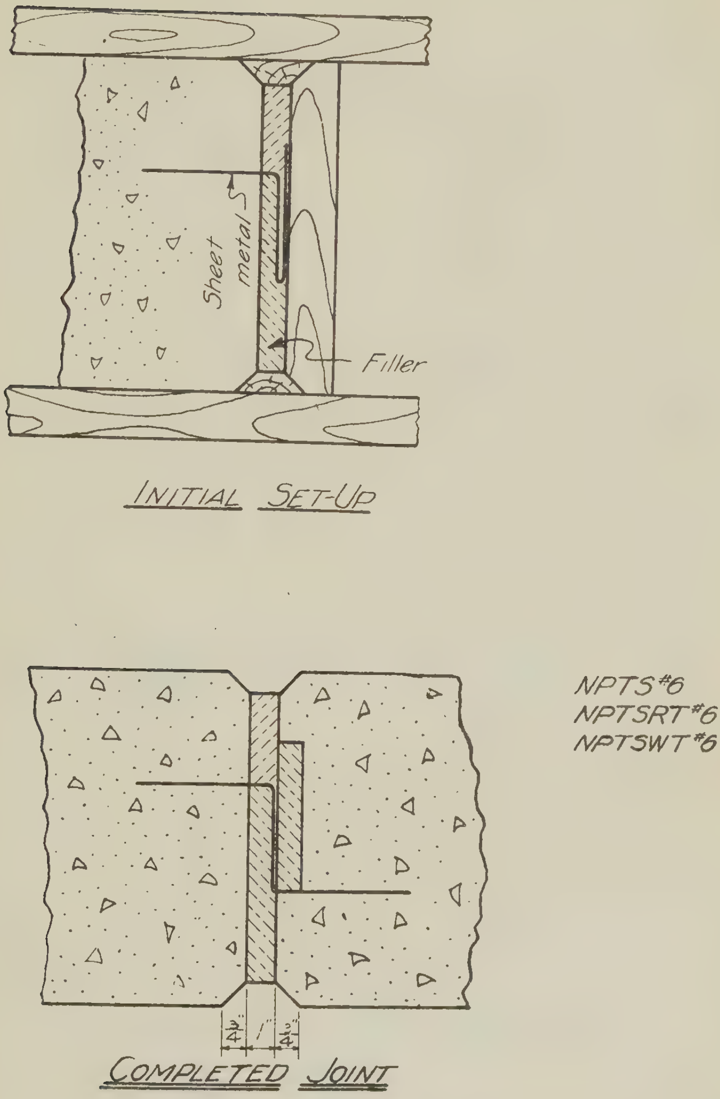

Sheet metal
Filler
INITIAL SET-UP
NPTS #6
NPTSRT #6
NPTSWT #6
COMPLETED JOINT

FIG. 6.

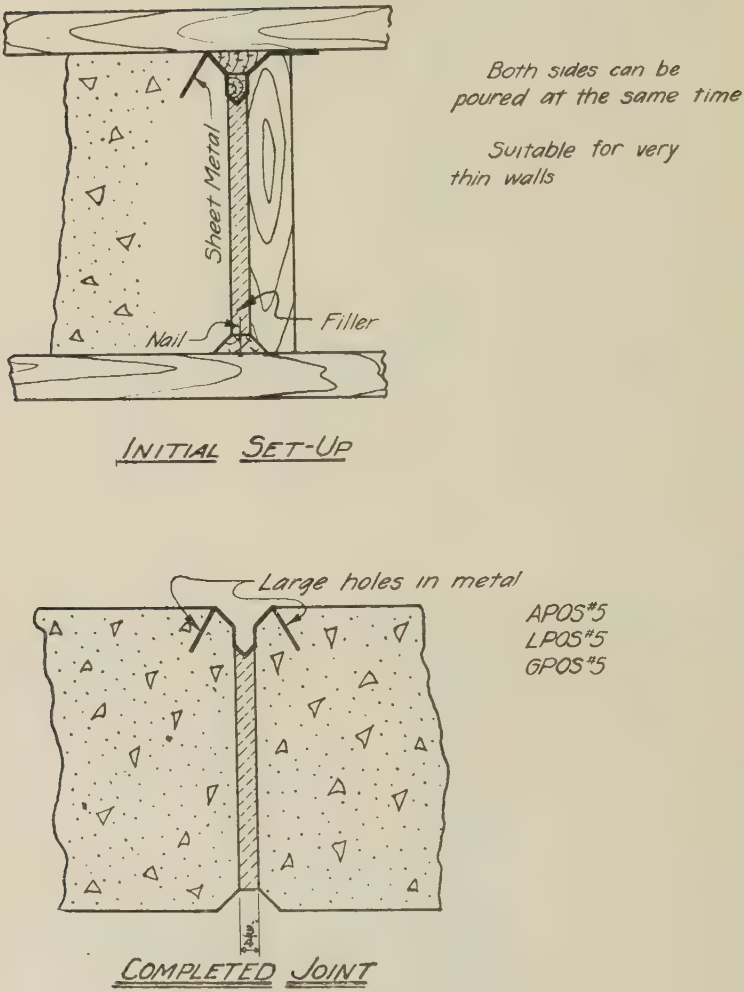

Both sides can be poured at the same time
Suitable for very thin walls
Sheet Metal
Filler
Nail
INITIAL SET-UP
Large holes in metal
APOS#5
LPOS#5
GPOS#5
COMPLETED JOINT

FIG. 7.

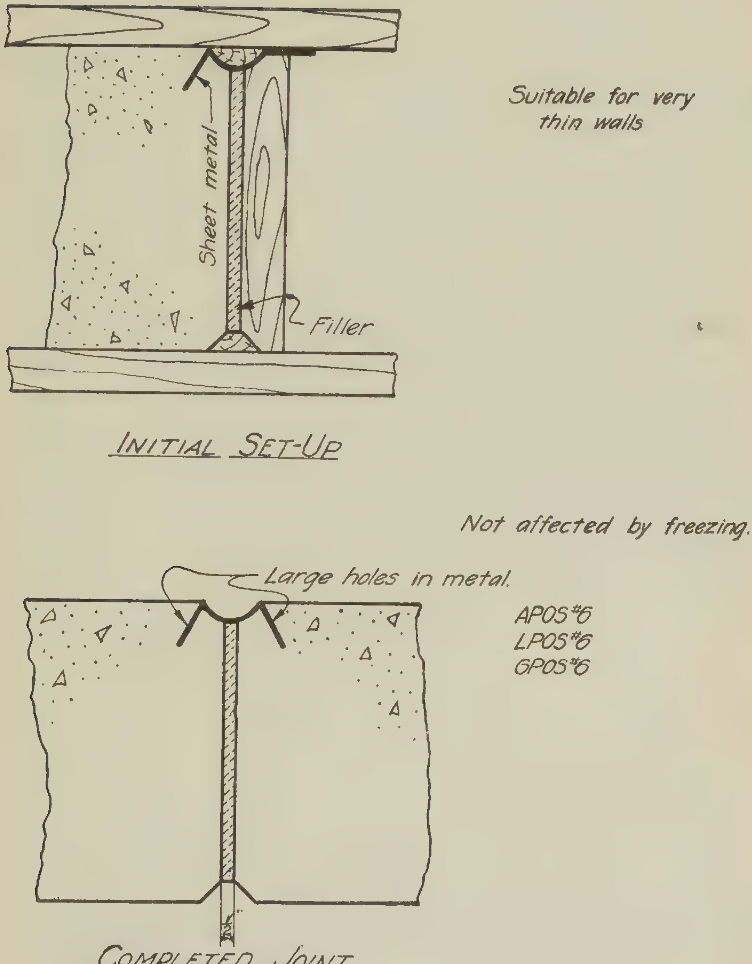

Suitable for very thin walls
Sheet metal
Filler
INITIAL SET-UP
Not affected by freezing.
Large holes in metal.
APOS #6
LPOS #6
GPOS #6
COMPLETED JOINT

FIG. 8.

Groups A and B demand that the joint have structural strength to resist external forces.

Group C need only have enough strength to endure the stresses caused by the to and fro movements of the parts it is between.

For Groups A and C a pliable bent copper sheet of the usual form might be sufficient.

For Group B, since it is subjected to pressure from both sides, two pliable bent copper sheets of the usual form placed thus might possibly be adequate.

A joint can belong to two classes., i. e., Liquid pressure on one side and Granular pressure on the other side. Such a joint is designated *LPOS, GPOS.*

Classes APOS, APTS, NPTSWT demand that the joint extend around the entire periphery (except in some cases, at the bottom).

The following pages show sections of different types of expansion joints. Each type under each class is given a number and the designations is APOS No. 1, APOS No. 2, or LPTS No. 1 as the case may be.

This material is submitted in no way as a complete report, but merely as a series of suggestions to put before the membership, in the hope that from it will result some discussion, suggestions, and, in particular, information based on the experience of the individual designers and constructors who compose the Institute. The committee has in mind for the continuation of the work a brief summary of existing structures and behavior of expansion joints so far as the same can be compiled, and we welcome, therefore, any data that the members can submit.

LANGDON PEARCE,
Chairman.
R. R. LEFFLER,
Secretary.

REPORT OF COMMITTEE P-6, ON CONCRETE PRODUCTS PLANT OPERATION.

ECONOMICAL MANUFACTURING PROCESSES.

The economical production of concrete products of uniform quality is a subject to which little attention has been directed by manufacturers or investigators. Exhaustive research work has definitely established the laws governing the more important properties of plastic concrete mixtures such as are generally used for monolithic concrete structures and concrete products of the so-called "wet cast" type. However, little is known regarding the application of these laws to the drier concrete mixtures used for tamped machine made products.

For the purpose of obtaining more information upon the factors which govern the strength and other properties of the relatively dry concrete mixtures members of this committee have carried out considerable experimental work in products plants.

The results of these investigations show that in general the fundamental laws governing the properties of plastic concrete mixtures may be applied to the drier concrete mixtures used in the commercial manufacture of concrete products.

It is the purpose of this report to point out how these fundamental principles may be applied to the manufacture of concrete products so as to reduce production costs.

A study of laboratory and field tests shows that the principal factors controlling the quality of concrete are:

(1) *Aggregate.*
(2) *Quantity of Cement.*
(3) *Quantity of Mixing Water.*
(4) *Time of Mixing.*
(5) *Curing-age.*

Aggregate.—Products manufacturers as a rule do not know what constitutes a good aggregate, and are therefore unable to judge the relative value of the various available aggregates.

For maximum strength and durability the aggregate must be clean. It should be free from organic impurities * and should not contain more than 10 per cent silt by volume. To secure a given strength more cement is required with poor aggregate than with good aggregate.

Other conditions being equal that aggregate having the greatest percentage of coarse particles will make the strongest concrete provided it can be satisfactorily handled by the machine. The fine aggregate com-

* Refer to Circular 1—Structural Materials Research Laboratory or Vol. 19. American Society for Testing Materials Proceedings.

monly used produces a smooth appearing product at the sacrifice of strength. Such aggregate should be corrected by the addition of coarser particles. Coarsely graded aggregates result in products of rough texture. Where objection to rough textured products exists manufacturers should make a special effort to overcome this objection by educating their customers to appreciate the economies which result through the use of coarsely graded aggregate. In certain types of units having thin walls, such as tile, it is however essential that the aggregate have a sufficient amount of the fine particles to compact the concrete in the thin walls and to prevent

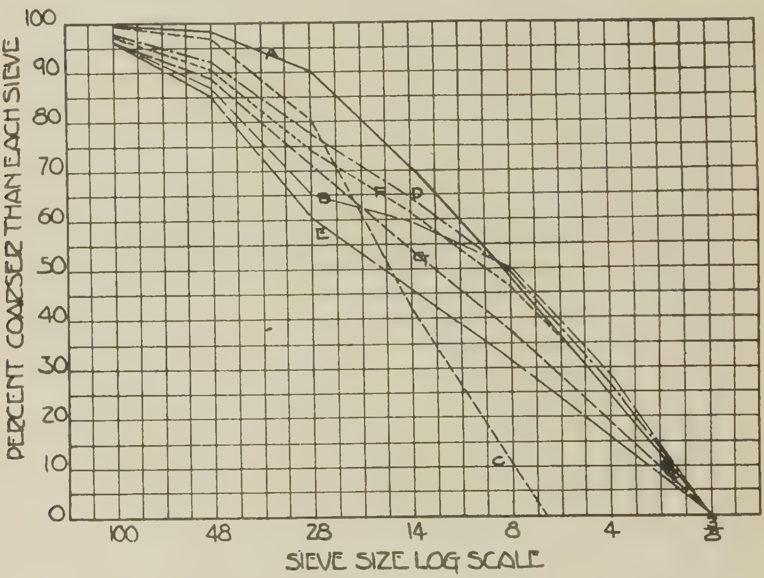


FIG. 1.—GRADING OF AGGREGATES USED IN BUILDING TILE TESTS.

the product from cracking or crumbling upon removal from the machine.

In a series of tests on building tile made at Cincinnati using a 1:4 mix gradings *B*, *E*, *F* and *G* in Fig. 1 gave mixtures best adapted to the type of machine used and produced tile of high strength; gradings *A*, *C* and *D* gave harsh-working unsatisfactory mixtures resulting in tile of rough appearance or relatively low strength. It is recommended that products manufacturers try out several gradings of aggregate to determine the coarsest grading it is possible for them to use with their particular machine.

The grading of an aggregate is determined by sieve analysis. A convenient measure of the grading of an aggregate is given by the fineness modulus.† The fineness modulus is the sum of the percentages of particles coarser than each sieve in the sieve analysis divided by 100 when the following size sieves are used: 100-48-28-14-8-4-⅜-¾-1.5-3.0.

Fine drift or beach sand graded to the No. 28 sieve has a fineness modulus of approximately 1.50. Medium sand graded up to the No. 8 sieve has a fineness modulus of approximately 2.50. Coarse sand graded to the No. 4 sieve has a fineness modulus of from 3.00 to 3.25. Roofing gravel graded from No. 4 to ⅜ has a fineness modulus of approximately 5.50. Pebbles graded from No. 4 to ¾ have a fineness modulus of approximately 6.25. Fine and coarse aggregates may be proportioned to give a combined aggregate any desired grading.

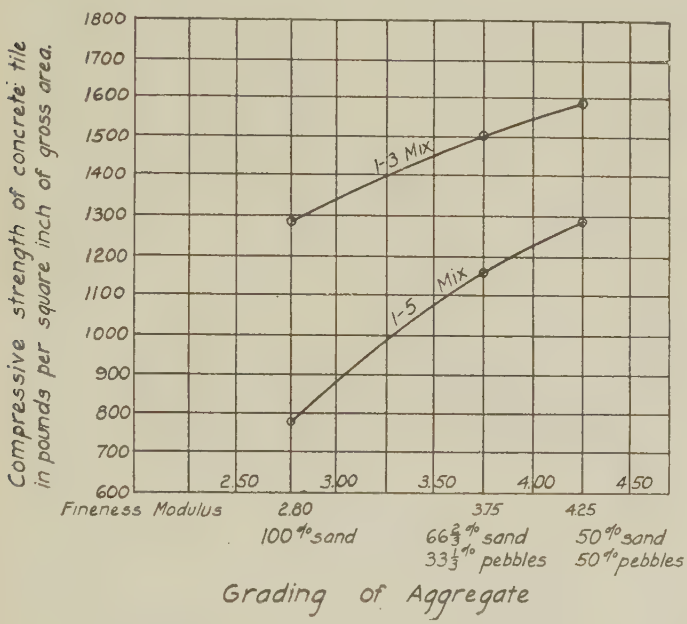


FIG. 2.—EFFECT OF GRADING OF AGGREGATE ON COMPRESSIVE STRENGTH OF CONCRETE TILE.

The effect of grading of aggregate, as measured by fineness modulus, on the strength of concrete tile and on concrete brick is shown in Fig. 2 and 3. It will be noted that as the percentage of coarse aggregate increased, resulting in a higher fineness modulus, there was a marked increase in strength, regardless of the mix. Tests show, however, that there is a limiting value for the fineness modulus depending upon the maximum size of the aggregate, proportion of cement, size and shape of product and machine used. This maximum value of fineness modulus can only be determined by experience.

† Refer to Bulletin 1—Structural Materials Research Laboratory.

Fig. 2 shows that the same strength was obtained with 1: 5 mix using aggregate having a fineness modulus of 4.25 as was obtained with a 1 : 3 mix for a fineness modulus of 2.80. Similarly in Fig 3 the same strength was obtained in a 1: 9 mix having a fineness modulus of 4.25 as was obtained for the 1: 5 mix with fineness modulus 2.75. This demonstrates the

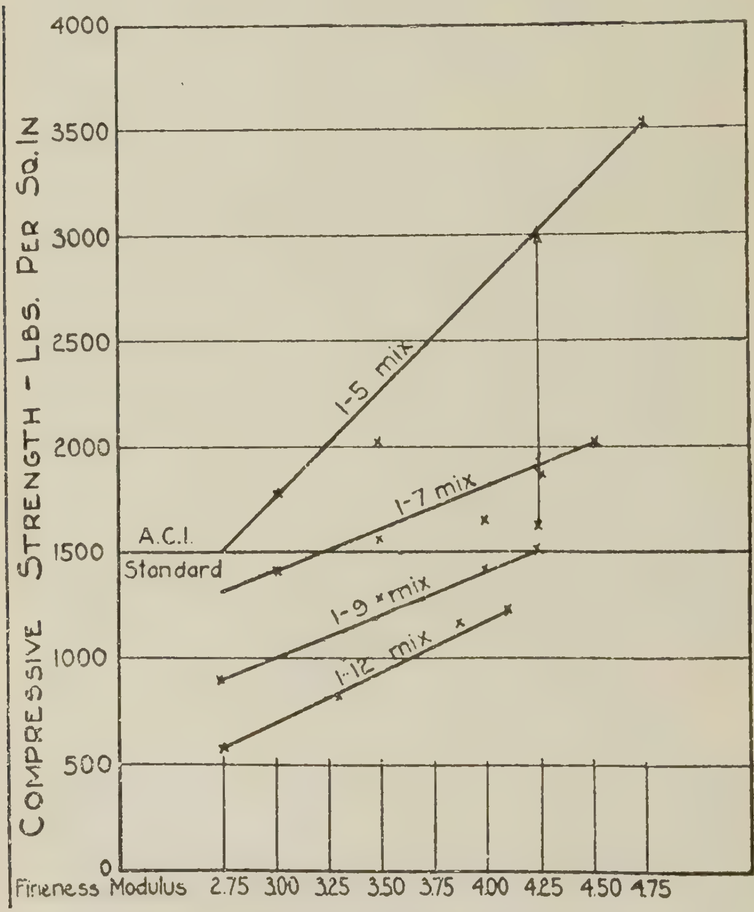


FIG. 3.—COMPRESSION TESTS OF CONCRETE BRICK.
Sand and Pebble Aggregate, 28 days.

saving in cement which may be effected through the use of the coarser aggregates. This saving may amount to as much as 2 cents for each 8x8x16-in. block. In a plant producing 300,000 block per year, this means $6,000 per year; the interest on $100,000 at 6 per cent. Such a saving will easily justify the purchase of better-graded aggregate and the equipment necessary for handling it.

Quantity of Cement.—Fig. 2 and 3 show that for the same grading of aggregate substantial increase in strength can be obtained with increase in richness of the mix. The results of these and other tests indicate that for each 1 per cent of cement that is added there is an increase in strength of approximately 1 per cent for the mixtures ordinarily used in the manufacture of concrete products. This will enable the manufacturer to adjust proportions to obtain the required strength and also to judge of the relative economy of available aggregates.

Amount of Mixing Water.—Tests show that the strength of concrete is influenced to a marked degree by the amount of mixing water used. Either too much or too little water will decrease the strength materially. For wet-cast products the highest strength is obtained when the amount of mixing water used is reduced to the smallest quantity that will provide a workable mix. Such a mix will produce a dense, compact, finished product without pockets or "honeycombing." The use of vibrators or other mechanical devices will assist in producing such a product with less mixing water than ordinarily employed. For tamped machine made products the highest strength is obtained when the amount of mixing water used is increased to a quantity sufficient to provide a workable mix. Workability for tamped machine-made products may be determined by compacting a ball of concrete with the hands for about ½ minute. Concrete of the proper workability will show a thin film of moisture on the surface of the ball after this treatment. Another indication of the proper workability is the appearance of water marks on the stripped surfaces of the unit. The effect of amount of mixing water on the strength and absorption of tamped machine-made block is shown by the data in the following table:

TABLE 1.

Mix 1:5 by volume, aggregate sand and pebbles 375 Fineness modulus.

Consistency.	Water, lb. per Sack of Cement.*	Compressive Strength, lb. per sq. in.		Absorption, Per Cent Gain in Weight after 24 Hours.
		Gross Area.	Net Area.	
Wet	59	1125	1908	5.5
Damp	51	1025	1732	5.6
Dry	43	917	1550	6.8

* Includes moisture contained in aggregate.

The above "wet" consistency produced a product with a distinct water web and fulfilled the conditions already described for a workable concrete. It will be noted that this consistency gave the highest strength and lowest absorption.

Mixing.—To secure maximum uniform strength of the concrete the materials must be thoroughly mixed. Results of tests made at the Structural Materials Research Laboratory, Lewis Institute (*Proceedings*, A.C.I.,

1918) show that increase in time of mixing materially increases the strength and uniformity of the concrete. Fig. 4 shows the relation between the time of mixing and the compressive strength at 28 days for relative consistencies ranging from 0.90 to 2.00. The driest concrete used in these tests (Relative Consistency 0.90) is about the same as that which may be employed for tamped machine-made products. For wet-cast products a consistency of about 1.10 would ordinarily be required. The curves

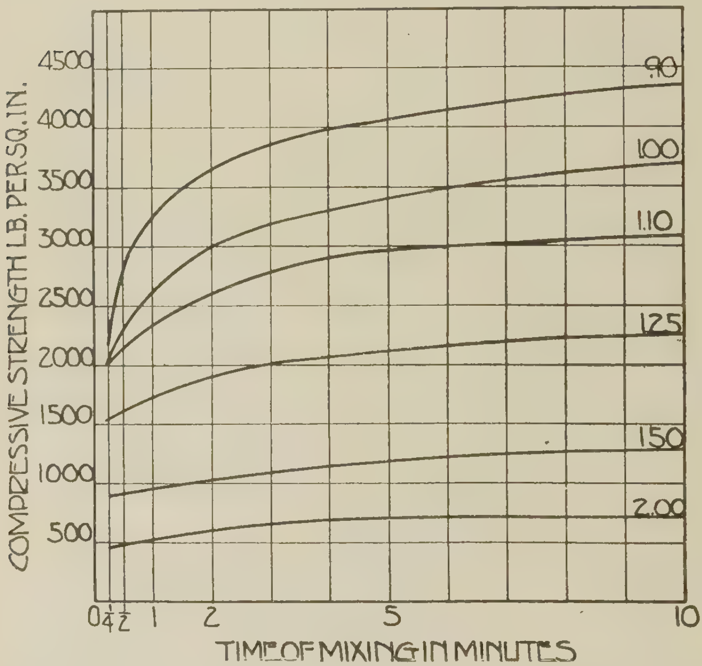


FIG. 4.—THE EFFECT OF TIME OF MIXING ON THE COMPRESSIVE STRENGTH OF CONCRETES HAVING VARIOUS RELATIVE CONSISTENCIES, AGE 28 DAYS.

show that substantial increases in strength are obtained from increased time of mixing and that greatest increases are obtained for the drier consistencies. This shows that more benefit is gained by increasing the mixing time of drier mixes. Table No. 2 gives the percentage increase in strength at 28 days obtained for concrete of relative consistency .90 and 1.10. As stated previously these consistencies approach those best suited for dry tamp and wet-cast process respectively. It is evident from these data that it is desirable to mix at least 2 minutes or longer. The time of mixing is calculated after all materials, including water, are in the mixer. Increasing the speed of the mixer does not have much effect on the strength

of concrete. Consequently nothing will be gained by speeding up the mixer in an effort to increase its production.

Curing.—The maximum advantages from the control of aggregates, quantity of cement, quantity of mixing water and time of mixing can only be secured when the curing conditions are also controlled. Tests show that the full strength of the cement is only developed when concrete hardens in a warm moist atmosphere. Under such curing conditions the strength at 28 days will be practically the same for temperatures of from approximately 65 to 212 deg. Fahrenheit. But the strength of the concrete at from 1 to 7 days is materially increased by hardening at the higher temperatures.

TABLE 2.—EFFECT OF TIME OF MIXING.

Percentages of strength of concrete mixed 2 minutes:
Consistency 0.90 for tamped concrete.
Consistency 1.10 for wet cast concrete.

Relative Consistency.	Time of Mixing, minutes.							
	¼	½	1	2	3	4	5	10
0.90	58.8	75.5	89 4	100 0	105.4	109.1	111.2	120 0
1.10	78.5	82.6	89.4	100.0	107.7	112.5	114.5	119.3

TABLE 3.—EFFECT OF TEMPERATURE OF THE CURING OF CONCRETE.

Percentages of strength at 28 days curing at 65° F.

Length of Curing, days.	Temperatures. Degrees Fahrenheit.			
	65	95	125	212
1	12	22	35	44
2	21	35	49	59
3	32	44	60	68
7	56	68	79	85
28	100	103	109	103

Above is based on curing in saturated atmosphere.

As the manufacturer should endeavor to produce products that will always pass definite requirements, but with a uniformly minimum margin, he must know that the curing conditions are practically the same at every season of the year. Such curing conditions are best obtained by providing rooms for curing where the amount of heat and moisture can be controlled in any desired combination.

The manufacturer who finds it necessary to deliver products at less than 28 days after they are manufactured can be guided by table No. 3 which shows the strength of products hardened at various temperatures in a saturated atmosphere. The strengths are given as percentages of the strength of products hardened in saturated atmosphere at 65 deg. Fahren-

heit for 28 days. (Data from tests made at Structural Materials Research Laboratory. 6 x 12 in. cylinders cured in water at various temperatures.)

Conclusion.—The recommendations in this report are emphasized because the time is not far distant when the manufacturer must take advantage of every production economy in order to survive the keen competition of the more progressive concrete products manufacturers and producers of other building materials.

J. W. LOWELL, *Chairman*,
Committee P-6,
Concrete Products Plant Operation.
E. W. DIENHART, *Secretary*.

The Committee wishes to express its appreciation of the co-operation of the following organizations that took part in the investigations referred to in this report:

Portland Cement Association
Structural Materials Research Laboratory
Universal Portland Cement Co.
Marquette University, Milwaukee, Wis.
Drexel Institute, Philadelphia, Pa.
Prairie Concrete Products Co., Prairie du Chien, Wis.
Wm. H. Devos, Milwaukee, Wis.
South Philadelphia Builders Supply Co., Philadelphia, Pa.
Hilker Supply Co., Granite City, Ill.
Stainfield & Nichols Co., Joliet, Ill.

[Discussion of this report is included in the discussion on Products Plant Problems, p. 546 of this volume.]

REPORT OF COMMITTEE E-7, ON WATERPROOFING.

During the past year Committee E-7 has held two meetings of the main committee. The sub-committees, however, have been rather active in collecting data and reviewing the literature on the subject of waterproofing concrete. As stated in our 1922 report to the Institute, the bases upon which the work of the committee would be conducted are as follows:

(1) That the committee take up the subject of waterproofing from the standpoint of performance of materials as determined by tests.

(2) That the committee consider the method of application of the waterproofing material to the structure, to be presented in the form of a specification.

(3) That the committee consider the waterproofing details exclusive of structural design and present same in the form of a recommended practice.

(4) That the committee formulate recommended practice for producing watertight concrete without the use of integral compounds or membranous or surface treatments.

(5) That the committee consider and recommend general principles which might control types of waterproofings to be applied to particular jobs of construction, same to be presented in the form of a recommended practice.

The present organization of the committee includes the following sub-committees:

Sub-Committee I—Integral Methods and Plaster Coatings

A. B. Cohen, *Chairman*
A. W. Stephens
D. A. Abrams
W. A. Freret
Joseph Rose

Sub-Committee 2—Penetrative Surface Treatments

E. H. Berger, *Chairman*
A. E. Horn
M. O. Withey
J. C. Pearson
Joseph Rose

Sub-Committee 3—Bituminous Coatings and Membrane Systems

G. L. Lucas, *Chairman*
Ernest Ashton
E. H. Berger
C. N. Forrest
S. R. Church
Max. Toch
R. H. Krecker

Sub-Committee 4—Non-Bituminous Surface Coatings

Hermann Fougner, *Chairman*
Ernest Ashton
J. C. Pearson
A. E. Horn
J. H. Libberton

Sub-Committee 5—Waterproof Concrete

A. W. Stephens, *Chairman*
D. A. Abrams
H. Fougner

SUB-COMMITTEE 1—INTEGRAL METHODS AND PLASTER COATINGS.—Sub-Committee 1 has been compiling all the available information and data upon the use of inert and chemically active integral compounds to waterproof concrete, and also upon the effectiveness of plaster coatings with and without the addition of integral compounds as outlined in last year's report. This work, however, has not progressed sufficiently to warrant any definite recommendations.

SUB-COMMITTEE 2—PENETRATIVE SURFACE TREATMENTS.—As stated in last year's report the work of Sub-Committee 2 is limited to those products which penetrate into the surface of the concrete filling the pores and thus rendering the surface impervious to moisture. These compounds may be inert and owe their effectiveness to their pore or void filling properties, or they may react with constituents in the surface treated forming compounds of greater volume, pore-filling capacity, and more insoluble in water. Compounds, the chief value of which depends upon their ability to form surface films, are not included.

During the past year Sub-Committee 2 has been reviewing the literature and collecting all available information on the use and value of penetrative surface coatings for waterproofing concrete. Most of the work of the sub-committee has been carried on by correspondence, but one meeting was held on December 18 at which the work of the past year was summarized and plans for the coming year made.

From our review of the literature it was found that a number of materials not included in our last year's report and coming within our scope has been used for waterproofing concrete. The information given in the literature is very meager and covers chiefly the methods of application and

the supposed chemical behavior of these materials when applied to concrete, limestone, and sandstone. Very little data as to the success of these treatments was found.

The classification of the materials used as penetrative surface treatments given in our last year's report has been amplified, as follows:

(1) Water solutions of inorganic salts which are supposed to react chemically with constituents in the concrete.

(a) *Magnesium, Zinc, Lead and Aluminum Fluosilicates.*—The fluosilicates are supposed to react chemically with the calcium hydrate and calcium carbonate present in the concrete and to precipitate in the pores of the treated surface insoluble fluorides, carbonates, and silicic acid. The fluosilicates are sold either in the solid or crystalline form or as water solutions containing from 5 to 15 per cent of the salt. The magnesium salt is usually used either alone or in combination with the zinc salt. The method generally *employed* for this application is to use three coats or treatments and to allow sufficient time between treatments for the complete evaporation of the water. In most cases a 5 per cent solution is *used* for the first treatment and 10 to 15 per cent solutions for the second and third treatments respectively.

(b) *Sodium Silicate (Water Glass).*

(c) *Zinc, Iron and Aluminum Sulphates and Alum.*

(d) *Barium and Calcium Chlorides.*

(2) Water suspensions of substances or mixtures of substances of a pore-filling character or which react chemically with each other or with constituents in the concrete and are supposed to form pore-filling compounds.

(a) *Iron Filings and Ammonium Chloride (Sal Ammoniac).*—This mixture usually consists of powdered or granulated cast iron with from 2 to 10 per cent of ammonium chloride. As a surface treatment it is mixed with water and several coats are applied by brushing. The reaction which takes place ultimately produced iron oxide or hydrated iron oxide in the pores of the treated surface.

(b) *Casein.*—A solution in ammonia water of casein alone or mixed with other substances is the usual form in which casein is used. The principle of the treatment is supposed to depend upon the formation of insoluble casein compounds.

(c) *Fuel Oil and Soap.*—This treatment consists generally of an emulsion of fuel oil and soap in water.

(3) Soaps.

(a) *Water Solutions of Alkali Soaps.*—These usually consist of water solutions of the oleic and stearic acid soaps of sodium, potassium, and ammonium. They are supposed to react with the lime hydrate in the concrete, forming insoluble water repellent lime soaps.

(b) *Solutions of Metallic Soaps in Volatile Solvents.*—These mate-

rials consist of calcium, aluminum, magnesium, zinc, and iron soaps mixed with small amounts of rosin, resins, or fatty oil and dissolved in turpentine and petroleum and coal tar distillates. Their value is supposed to depend upon the deposition of the soaps in the pores of the concrete upon the evaporation of the solvent.

(4) Combinations of solutions, in two or more applications, which are supposed to react chemically with each other in the pores of the concrete, filling them with substances of a water repellent or insoluble character.

(a) *Soap Solutions and Inorganic Salts.*—The well-known Sylvester's process for waterproofing concrete is of this type. It usually consists of a solution of 2 oz. of alum in one gallon of water, and a solution of 12 oz. of some soap in a gallon of water. The concrete surface is first coated with the alum solution and when this has been absorbed the soap solution is applied over it. The chemical reaction which takes place results in the formation of an aluminum soap.

(b) *Inorganic Salt Solutions.*—Two treatments of this type have been used. The first consists of applying a water solution of sodium sulphate or alum to the concrete surface and when it has been absorbed to apply over it a solution of barium chloride. In this way insoluble barium sulphate is supposed to be precipitated in the pores. The second consists of a similar treatment but using sodium silicate and calcium chloride.

(c) *Sodium Silicate and Acid Solutions.*—This treatment consists in applying the sodium silicate solution first and when this has been absorbed applying a dilute solution of an acid such as hydrochloric, tannic, and phosphoric over it.

(5) Solutions of Liquid and Solid Hydrocarbons.

These materials consist of heavy petroleum distillates such as lubricating oil, petrolatum, etc., or paraffin dissolved in volatile solvents such as mineral spirits, gasoline, coal tar naphtha, and turpentine. The solutions contain from 1 to 15 per cent of solid matter which is supposed to be deposited in the pores of the concrete upon the evaporation of the solvent.

(6) Waxes, Fats and Solid Hydrocarbons Applied Hot.

This group of materials consists of waxes of various kinds alone or combined with fats and fatty oils, fats, and solid hydrocarbons such as paraffin and ceresin, which are applied in the molten condition and are then driven into the pores by the aid of heat.

Efforts have been made to obtain information concerning the value and effectiveness of penetrative surface treatments from architects and engineers but with little success. The sub-committee, however, intends to continue its efforts along this line and have tentatively agreed upon the following form of questionnaire to be sent out:

(1) What has been your experience concerning the permeability of concrete in superstructure work?

(2) If you have had difficulty in obtaining impermeable concrete, what methods and materials have you used to overcome it?

(3) Have you used penetrative surface treatments for this purpose, as magnesium fluosilicate, etc.? If so, what materials have you used, how were they applied and with what results? Can you give locations of the jobs?

In view of the apparent lack of dependable information upon the use and effectiveness of these materials, the sub-committee has decided to conduct experiments to obtain this data. The exact form which this investigation will take has not been definitely decided but this will be agreed upon shortly.

SUB-COMMITTEE 3—BITUMINOUS COATINGS AND MEMBRANOUS SYSTEMS.—The work of Sub-Committee 3 has been held up temporarily until the specifications for waterproofing and dampproofing materials now being discussed by Committee D-8, A. S. T. M., have been agreed upon. So far the sub-committee has decided upon the various types of materials to be considered under the subject of dampproofing and the form which the specifications and practice they recommend will take. The materials being considered under dampproofing are as follows:

Dampproofing Materials

- (a) For cold application
  - (1) Above ground level
    - (a) Paint or solution for stone backing to prevent staining of exposed surfaces
    - (b) Dampproof coatings for application to the inside of exterior walls to prevent staining of plaster
    - (c) Primer for use under hot asphalt
    - (d) Primer for use under hot coal tar pitch
  - (2) Below ground level
    Dampproof coatings for application to the exterior surface of walls
- (b) For hot application
  - (1) Above ground level
    - (a) Asphalt over primer
    - (b) Coal tar pitch over primer
  - (2) Below ground level
    - (a) Asphalt over primer
    - (b) Coal tar pitch over primer

The form which the specifications and recommended practice will take is a follows:

- (1) General information
- (2) Specification for material
- (3) Application
  - (a) Preparation of surface
  - (b) Method for applying material, equipment, etc.
  - (c) Protective covering

The sub-committee has considered the section entitled "Tentative Standard for Bituminous Waterproofing" in the report of Committee S-2 for 1922 and recommends that it be withdrawn from the records of the Institute. This tentative standard is not a definite specification but is in the nature of a description of the points to be considered in drawing up a specification. While the view expressed are in general correct, the information given is not definite enough to be of material assistance to engineers and architects.

SUB-COMMITTEE 4—NON-BITUMINOUS SURFACE COATING.—Sub-Committee 4 has held several meetings and early in its discussion it was decided that the scope of Sub-Committee 4 would be limited to those products which form surface films in counter-distinction to those which have pore-filling but not filming qualities.

Under the above main division it was decided that the sub-committee consider its program under the following sub-division:

(1) Cellulose compounds
(2) Drying and semi-drying oils
(3) Admixtures with the drying and semi-drying oils of other materials for special qualities
(4) Pigment admixtures with the above—generally called paints
(5) Solid materials which require heating for application
(6) Solid materials thrown into solution by solvents of various kinds
(7) Metallizing processes

In the past year various authorities have been consulted and data investigated to a considerable extent. So far, the sub-committee is able to report at this time only sub-divisions *1* and *2* as follows:

(1) *Cellulose Compounds.*—The objection to cellulose compounds is due to the fact that rapid evaporation of the solvents causes the formation of a film with but little penetration. In addition, after years of exposure, the film has a tendency to disintegrate and peel. Consequently, success in the use of such compounds in the light of present experience is doubtful. Tests, however, under way at this time indicate that it may be possible to reduce the viscosity and increase the solid content in cellulose compounds. If this can be accomplished, further experiments are recommended with this class of material.

(2) *Drying and Semi-Drying Oils.*—Drying oils are those which dry by oxidation within a period of a few weeks. In this class would be included such oils as linseed, perilla, sunflower, chinawood, soya bean, etc.

From the initial investigation, these oils possess value for a considerable period of time, and where applied to concrete surfaces which have had ample opportunity to carbonate will give service equal in every way to their similar use on other types of surfaces.

Semi-drying oils are those oils which take a considerable period of time to oxidize thoroughly. In this class would be included such oils as menhaden, shark, herring, whale, etc.

Since these oils require a considerable period for drying the film remains tacky, making possible the adhesion of dust particles which become a serious objection to their use.

The sub-committee is continuing the active study of the other subdivisions of its work and expects to have a more complete report by next year.

SUB-COMMITTEE 5—WATERPROOFING CONCRETE.—The work of this committee is being limited to producing waterproof concrete by a combination of portland cement and water with fine and coarse aggregates. Concrete in which there is a material other than portland cement and the materials ordinarily used as fine and coarse aggregates will not be included in the work of the committee.

While the sub-committee has been collecting data on this subject it has no definite recommendations to offer this year.

The work of Committee E-7 during the past year has been chiefly in collecting and compiling data upon the materials and methods now in use for waterproofing concrete, and the only definite recommendations it has to make are those offered by Sub-Committee 4 concerning the use of cellulose compounds and drying and semi-drying oils. Some research work has been planned by Sub-Committee 2 on Penetrative Surface Treatments, on account of the very meager and apparent lack of dependable information on these materials.

The committee has considered the section of the 1922 report of Committee S-2, Reinforced-Concrete Highway Bridges and Culverts, on "Waterproofing, Expansion and Construction Joints and Drainage." While this report gives general information on the subject of waterproofing, it is so indefinite as to be somewhat misleading, and inasmuch as this subject is within the scope of Committee E-7 and is being actively studied by it, Committee E-7 requests that it be withdrawn from the Standards of the Institute.

E. H. BERGER, *Secretary.*

REPORT OF COMMITTEE C-1, ON CONTRACTOR'S PLANT.

The purpose of this paper is a general statement of the principles on which plant design depends and to give as thorough an exposition of these principles as is possible to present at this time, this without any restriction as to the type of project under consideration, but to illustrate particular types of concrete plants applicable to a certain set of given conditions. There is to be no discussion as to the relative merits of any type of equipment as applied to general uses, but rather to consider the use to which each particular device may be put so that it will work out to the best advantage.

Plant design involves not merely the question of equipment but also of men, and the proper combination and co-ordination of the two. An estimate sheet may refer to materials and labor but any operation plan which does not consider the human element as being of major importance is faulty. Our problem properly stated is one of materials and men. An effort will be made to show just how these two factors can best be combined to attain the desired results on any concrete structure.

Construction of any sort is essentially a manufacturing proposition and the elements which enter into the design and layout of machinery in a fixed manufacturing plant are the same as those which enter into the design and layout of a plant for construction purposes,—with this difference—plant on a construction job is temporary and the ease of dismantling is to be considered as much as the cost of erection. Just as in the factory we have to take raw materials or their derivatives and change into the finished product, so on any piece of construction work we are to convert various materials into roads, bridges, dams or buildings of the quality desired and specified with the requisite speed and at a cost which is to be reasonable and just. There may be exceptions to this general rule where political or financial considerations take precedence over those stated but such jobs are luckily few and far between.

Knowing then these considerations entering into any construction work it is next necessary to obtain certain preliminary information and also, to carefully weigh various matters having to do with the efficient operation of the plant. We will endeavor to outline all these items and to show the influence exerted by each in making a proper design. We will give illustrations of various plants showing the application of the principles enumerated and how they fitted in with conditions which were found at the start of the job and how well they aided in the completion of the project as originally planned and scheduled.

The items entering into such a study have for convenience and reference been outlined in the form of tables. An analysis consists of two distinct steps which are separated as shown in the two tables attached:

Table "A"—Information desired, and

Table "B"—The various points to be considered in detail before making final decision on plant, with the various sub-headings under each of these major sub-divisions as shown in the outline.

TABLE "A"—INFORMATION DESIRED

1. Quantity materials needed—general: steel; lumber, brick and tile; cement; sand and gravel
2. Source raw materials or derivatives.
3. Location of these materials in finished structure.
4. Methods of transportation available.
5. Methods of handling that are available.
6. Space available—storage: traffic conditions; weather conditions; labor conditions
7. Power available.
8. Layout of ground.
9. Probable weather conditions.
10. Probable labor conditions.
11. Special consideration: Sequence of work; Relation to other work; Finished floors; Size sections

A - - - As previously outlined, the first step involved in design of plant is to obtain necessary information either from the engineer and owner, from plans or from a survey of the property and surroundings as they exist previous to the start of the proposed improvement.

1 - - - The first information necessary is the quantity and grade of each material entering into the finished project and in general consists of steel, lumber, cement, sand, gravel or stone, brick and tile. The quantities are necessary in order to determine how much storage space is necessary, the relative importance of each to the job, and also, in order to determine whether large or small scale production methods may be used,—in other words, how much may be economically expended on plant to save a certain amount per unit of material in handling. It is necessary to know the quality of material to be used so that methods of handling may be devised which will not be detrimental.

2 - - - The second item of information necessary is the source of raw materials or their derivatives. In being changed from their natural state to their condition in the finished work materials may undergo several operations. Whether these operations are performed by the builder or by the dealers previously handling the material depends largely on the size of the job and its accessibility. For instance, the savings on a large job, difficult of access and convenient to materials in their natural state would pay for the installation of machinery necessary to carry out the operations necessary to make all the transformations. On the other hand, a small job located centrally in a town would only justify the buying of materials in as nearly a finished state as possible from the nearest dealer. Between these two extremes lie many cases where a compromise must be made, the nature of which depends entirely on a close study of each individual case.

3 - - - It is necessary to know the location of all the materials in the finished structure. We may then lay out the plant and attendant storage so that each operation involving motion, places material more convenient for its final incorporation into the finished job. In this way handling may be done more easily and economically and also storage space may be conserved by placing all materials so that they will not interfere with the movement of other material which must reach their place in the structure at an earlier date.

4 - - - Closely allied to the points previously given are the methods of transportation available in any given section. It is necessary, for instance, to know whether freight cars, auto trucks, scows or other means are available, whether the job justifies it and if not, what other arrangement can be made by which material will be moved another step nearer its final place.

5 - - - With methods of transportation also goes the mechanical means available for handling material as it is useless to consider transportation methods without considering also how materials are to be loaded and unloaded. Naturally on small jobs mechanical methods of handling cannot be considered except in certain special cases where someone else may have equipment installed which would otherwise be idle and will perform the service quite cheaply on that account.

6 - - - No decision on methods of handling materials can be properly made without a survey of the site and surrounding features. A study of contours and occurrence of water is of special importance as the slope of the ground may, to a considerable extent, determine the design of the plant since all grades should be used to advantage. Water must be particularly considered where a tunnel or a mixer and tower pit is planned on and means must be devised to overcome this handicap with as little expense as possible, both in installation and operation. The extent of available storage space often determines methods since it will determine the size of lots in which material may be received.

7 - - - Power if available near the site, must be carefully investigated and all conditions applying thereto looked into to determine source for use on the job. Local rules of power companies as to types of motors to be used, etc., may often cause rejection of the most apparent source. Water also should be looked up at the same time.

8 - - - In case the builder is not familiar with the district where the construction is to take place it will be necessary to study the probable weather conditions during the course of the job. Where a job is to go ahead during the winter when cold and bad weather is probable the plant should be laid out so as to take advantage of every day not of extreme severity. This may be done by compacting the layout so protection is afforded to practically all the men required to operate. A balance must be struck here, however, between the inefficiency of the workmen under these conditions and the cost of a shutdown with attendant loss on overhead and lengthening of the job. Plants should also be designed larger for winter work than in summer in order to keep nearer to summer speed by accomplishing more on the fewer good days.

9 - - - Probable labor conditions during the course of the job must also be looked up. With low priced labor of the pick and shovel variety, less money may be spent on plant. If, however, labor is scarce and therefore high priced and independent, plant must be designed to eliminate all hard work and also as many men as possible. The attitude taken by various labor men in some sections may also influence a decision on certain types of equipment.

10 - - - Special considerations may be called for by plans and specifications in some jobs or may be desirable due to requirements of the architect, engineer or other interested party. Sequence of work may be required, relation to other work may be specified or structural conditions may be such as to materially effect plant particularly as to capacity. As an example, a specification calling for grit in monolithic finish floors demands a larger plant and an arrangement for handling and storing this material convenient to mixer. A large plant is then also required so that concrete may be completed earlier, finish put down and excessive overtime avoided on troweling. Size of concrete structural sections determines size of aggregate that may be used and, therefore, to a certain extent the size and design of plant. Small sections cannot be poured as rapidly as larger, therefore, a smaller plant may be installed.

The second study is summarized in Table "B" following, with the various sub-headings and the successive steps in analysis as follows:

TABLE "B"—POINTS TO BE CONSIDERED in detail before making final decision on plant.

1. Costs of various installations
   - Rental
     - first cost,
     - salvage, interest
   - Installation and removal
   - Maintenance
   - Operation
2. Reserve supply
   - Weather
   - Labor
   - Traffic conditions
   - Convenience heating
3. Convenience in weather protection
   - cold
   - rain
4. Few bottleneck equipments to avoid conjestion or to replace in case of breakdown.
5. Accessibility for repairs.
6. Co-ordination balancing all facilities.
7. Uniformity of mix.
8. Proportioning.
9. Location so as to reach all parts work with all materials.

B - - - After the special information applying to each particular job has been compiled from various sources the other considerations which apply to all contracts must be weighed before a definite decision on plant can be made as follows:

1 - - - The total cost of each possible installation must be balanced against the savings in handling various units of material. This cost consists of rental, installation and removal, maintenance and operation.

2 - - - Reserve supply necessary is determined by weather, labor and traffic conditions, also in some cases, by convenience in heating. Enough material should be stored so that any temporary interference in line of communications whether by traffic, weather or labor conditions will not cause a job tie-up before adverse conditions can be remedied.

3 - - - Convenience in weather protection of plant whether from cold or rain has already been well covered. This study involves a balancing of the cost of a day lost against higher unit costs of working under protection. It is also necessary that materials themselves may be so compactly stored as to be easily protected and heated.

4 - - - By avoidance of bottlenecks is meant a layout such that if any man or appliance cannot function, progress on the job will not be impeded. In case of machinery which by necessity occupies such a critical position

on the job as to cause an entire shutdown in case of failure, it is well to inspect frequently, provide spare parts if practicable, ascertain where repairs may be quickly made, and also, arrange an alternate so that job may carry on at a higher unit cost temporarily. As to men, it is well to have as few as possible that are absolutely essential to the proper conduct of a job. Should this, however, prove unavoidable arrangements must be made to train another to serve in an emergency so as not to delay work.

5 - - - All machinery should be so placed that in case of a failure every part is accessible for repairs or in a smash that the whole machine can be removed.

6 - - - By co-ordination is meant the balancing of all facilities for handling materials so that they will all carry on the job at the speed planned on. For example, it is useless to have a mixer and gang capable of turning out twenty yards an hour unless supply and reserve give us that quantity or that twenty yards can be taken away from the machine per hour. Only in case of varying demands being made on a machine or gang should either be larger than the normal. This condition, however, often occurs in the receipt of materials on a job due to supply not being constant. As a rule, plant should be larger than desired rather than run the risk of not being adequate.

7 - - - In order to secure uniformity of mix, it is necessary to start at source of material and so arrange handling that it will not be segregated but rather better mixed than when received. Raveling may be minimized by proper handling.

8 - - - Proper proportioning is more easily attained by careful planning of plant at the start of a job than by any other method. In fact, it is often impracticable later to make a change in handling. By arranging facilities so that space and headroom is left for measuring boxes we can guarantee that mix is being turned out as intended. This may mean a saving in materials, at least the results are known and not approximated.

9 - - - Layout of plant so as to deliver to all parts of the work is particularly desirable in the cases of those jobs covering a large area. Both work and layout must be considered so that at every step materials may go forward as desired and no recent work, other material, or interruption in supply interfere.

Conclusions.—We have now reached the point where, having obtained necessary information and having in mind the various points to be considered we are ready to proceed to the actual design of plant. To do this intelligently it is necessary to trace all materials from source to final place in job, estimate all handling costs whether these be of material, labor, or the various plant charges and balance the advantages and cost of each method of handling against those of the other. Some must be approximated from previous experiences as this is the only source of information.

If at all possible, crude methods should be discarded no matter if they show up better because of effect on the morale and the workmanship as this is by no means a negligible factor.

Since this is to be a general paper and to apply to all concrete construction whether of roads, dams, piers, or buildings, it is necessarily couched in general terms. Also, in making the exposition on handling materials we must limit ourselves to the receipt of the material on the job since we could not consider such a broad scope as would involve quarrying of rock, crushing of stone, etc., although these processes may be carried out on comparatively small jobs. It is assumed, however, that any plant study to be complete must go back further than indicated in this paper.

Material is usually brought on a job by freight or industrial cars, scows, trucks or for special cases, on cableways. From the point of receipt there are usually three operations,—1st, conveyance to storage,—2nd, from storage to mixer and,—3rd, mixer to place of deposit in the structure. A diagram attached will show relation of each of these operations to a variety of machinery generally used for this purpose. Each operation may or may not involve the use of both equipment and men. It is highly advantageous to make a close study right here in order to, if possible, omit one or more of these operations on as large a volume of material as possible in order to cut costs. We will recommend various short cuts in operation as we proceed with the discussion.

In receiving by scows or freight cars we are confining ourselves to such cases as are landed close enough to be handled into storage without trucking. Scows and freight cars of aggregate are generally unloaded by grab bucket although some jobs may be large enough for the use of bottom dump cars on a trestle. In using a grab bucket, if possible, layout so that point of receiving and center of bin over mixer is on the same circle of circumference, this to avoid booming of derrick. It is also very often possible to handle materials direct to mixer thus saving Operation No. 2.

Industrial cars are often used on road jobs and may carry loose aggregate, or batch boxes to be unloaded by a crane or derrick.

Trucks may carry the batch boxes or loose aggregate. Those with loose aggregate may be backed upon a planked runway or trestle over a tunnel which will facilitate the second operation. Contour of ground will often favor this method.

Cableways will handle into storage in a manner similar to grab buckets. Scrapers may sometimes be used to pile aggregate over a tunnel or against a bulkhead.

RECEIVING - - - OPERATION No. 1—on lumber and steel may involve crane or derrick on large jobs. Cement may be handled by traveling belt conveyor, roller conveyor, derrick or by hand wheeling.

The tryout to be made by one of the railroads of unit containers for handling brick may revolutionize methods by using the same container for a number of brick from kiln to floor where used.

HANDLING - - - *OPERATION No. 2*—may be carried out as far as aggregates are concerned by industrial cars particularly in the case of tunnel under storage pile mentioned under Operation No. 1. A steam shovel or travelling crane may be used where storage covers more ground than may be reached by a stationary derrick. In some cases storage directly over a mixer may be filled by belts or buckets from a boot into which trucks dump; thus eliminating Operation No. 2.

Where conditions are unfavorable to machinery, hand shoveling into barrows or carts may be used. Even here, a close time study is necessary to show which method is preferable. Cement in Operation No. 2 is handled by belts, derricks or by hand similar to Operation No. 1.

With other materials Operation No. 2 is omitted.

PLACING - - - *OPERATION No. 3*—covering the handling of wet concrete from mixer to place may employ a great variety of mechanical devices but consists essentially of elevating or lowering and transportation. In

Material Received in	Equipment Used.					
	Operation No. 1, Receiving.		Operation No. 2, Handling.		Operation No. 3, Handling and Placing.	
Freight Cars....	Grab Bucket..		Industrial Cars.			
			Trucks.........		Industrial Cars.....	
Industrial Cars..	Crane.........		Cableways.....		Trucks....	
			Crane..........		Cableways.........	
			Grab Bucket...		Cranes.............	
			Belt Conveyor..		Derricks...........	
Scows..........	Belt Conveyor	Storage	Derrick.........	Mixer	Belt Conveyors.....	To Place
			Bucket.........		Chutes.......	
			Buggy.........		Tower (one or more)	
Trucks.........	Bucket........		Barrow....... .		Buggy.............	
			Scrapers.......		Barrow............	
			Steam Shovel . .			
Cableways......	Scrapers......					

larger buildings compactly built it generally consists of stationary mixer, tower and bucket with chutes or buggies for distribution. In buildings covering a large area or small buildings requiring low plant expenditure it may involve only a small mixer with a pole or tower with bucket both of which may be movable or stationary. The mixer itself may be mounted on a tower and fed by a skip, the tower being on rollers and readily moved along as the work progresses. Mixer and attendant derrick are sometimes mounted on auto chassis or caterpillars to secure greater mobility.

On large jobs covering a great area there is often a choice to be made between several unit plant installations or having one large central plant distributing concrete to various parts by cars or by chuting from top of one tower to base of another, etc.

In construction of concrete bridges floating equipment on which all three operations may be carried out may be used—the derrick for receiving, mixer with overhead hopper, cement shed with belt, tower and chutes all on the same barge.

On dams and piers it is possible to use cableways, industrial cars, flat cars in conjunction with derricks and cranes, belts and also towers and chutes.

On roads and pavements the mixer is usually movable and chutes or bucket are used to convey concrete directly into place.

For materials other than wet concrete the usual procedure is to so arrange the concrete plant that these materials can be handled with the tower equipment and at the same time, unless the job is of such a size as to require hand handling. Niggerheads on hoists with booms on towers or separate gin poles, travelling derricks, etc., will handle steel or lumber. Brick and tile will be handled by a hod hoist or by a platform in the concrete tower.

In special cases there will be other materials than those mentioned which will require attention in laying out. It is rarely indeed that a plant used on one job can be economically duplicated on another even though the jobs themselves are similar. A change in anticipated labor or weather conditions may force a material change in plans.

In designing, however, when at a loss to make a decision on the best method of handling it is well many times to come back to the original proposition: material is to go from source to its final place in the work as a geometrical proposition—a straight line is the shortest distance between two points. Modifications are naturally necessary but these may be made by judging the relative importance of the various materials entering into the structure and decision rendered accordingly.

4 - - - Exhibition of slides showing various plants and pointing out the application of principles previously given will be made at the time of submitting report.

(Signed) JOHN G. AHLERS, *Chairman.*
(Signed) W. F. LOCKHARDT, *Secretary.*

REPORT OF COMMITTEE P-4, ON CONCRETE STAVES.

The tentative specifications and building regulations for concrete staves, presented in this report, constitute the work of Committee P-4. These specifications are based on a study of the structural requirements of staves when used in the construction of silos, grain tanks, coal pockets and other buildings. They are also based on data secured from a comprehensive series of tests in which staves were supplied by representative manufacturers from different sections of the country; each manufacturer submitting six specimens representing his ordinary commercial product. The Committee wishes to make grateful acknowledgment for the assistance rendered by officials of the Structural Materials Research Laboratory of Lewis Institute, who made the tests and directed the test program. Acknowledgment is also made to the National Concrete Stave Silo Association and the various stave manufacturers who co-operated in these tests. Other tests made prior to the past year also furnished data used in the formulation of the proposed specifications. Special acknowledgment is made for the valuable data secured from the comprehensive series of tests made in the Peoria, Ill., plant of the Michigan Silo Co.

A brief summary of the results of the tests made at the Structural Materials Research Laboratory, Lewis Institute, are presented in Tables I and II. Table I gives information regarding the methods and materials employed in the manufacture of the staves. Table II gives some of the results of the absorption and strength tests.

The transverse tests were made in a 40,000-lb. Riehle universal testing machine when the staves were 28 days old. The staves were tested flat and were supported at one end by a ¾-in. round bar and at the other end by a small spherical bearing block; the distance between supports being exactly 24 in. The load was applied at mid-span through a spherical bearing block. Metal plates 2¾ in. square, shimmed with leather, were placed between the concrete and the supports and the loading block.

Compression tests were made on specimens cut from half staves after the transverse test; each being about 7 in. in height by 5 in. long and 2½ in. thick. The load was applied parallel to the 7 in. dimension. The ends of the specimens were capped with a mixture of portland cement and gypsum to insure an even distribution of the load which was applied through a spherical bearing block placed on top of the specimen. On account of the time required to chisel out the specimens, there was some variation in their age when tested, which ranged from 31 to 38 days.

Absorption tests were made on sections cut from half staves. The area of the largest face of each specimen was about 100 square inches.

The specimens were dried to constant weight and immersed in water at room temperature for the following periods: 3 minutes, 1, 3, 6, 24, and 48 hours. They were then surface dried with a towel. The increase in weight expressed as the percentage of the dry weight is the absorption.

A high transverse strength and a low absorption are highly desirable qualities in concrete staves. The transverse test indicates the ability of the stave to resist the outward pressure imposed upon it between the reinforcing hoops of silos, grain tanks, coal pockets and other circular structures used for storage purposes. A low absorption is important as it is a measure of the watertightness of the stave.

The compressive strength indicates the ability of the stave to resist forces which tend to crush it. When used in coal pocket or grain tank construction, the stave walls of the bins sometimes subjected to moderately heavy loads due to the weight of the head house sheltering the conveying equipment. In silos of moderate size, the load on the staves, due to the weight of the structure, is comparatively low. Staves that meet the re-

TABLE I.—MISCELLANEOUS DATA OF CONCRETE TEST STAVES

Lot No.	Process of Manufacture.	Method of Tamping or Vibrating.	Mix.	Aggregate.		Time of Mixing.	Type of Mixer.	Method of Curing.
				Fineness Modulus	Size.			
7142	Tamped	Machine	1:1½:1½	2.94* 4.27†	0 – 4 30 – 4	3 min.	Batch	Steam, 48 hrs. Sprinkled 10 days
7125	Cast	Vibrated	1:3	2.71* 5.37†	0 – 8 8 – ⅜	4 min.	Batch	
7112	Cast	Vibrated	1:5	3.69	0 – ⅜	2 min.	Batch	Steam, 24 hrs.
7080	Tamped	Machine	1:3	2.32	0 – 16	1 min.	Batch	Sprinkled 10 days
7109	Tamped	Machine	1:4	2.72	0 – 16	2 min.	Batch	Sprinkled 7 days
7107	Tamped	Machine	1:3	2.84	0 – 8	3 min.	Batch	Sprinkled
7123	Tamped	Hand	1:3	2.94	0 – 8	4 min.	Batch	Steam, 48 hrs.

* Fine aggregate.
† Coarse aggregate

TABLE II.—DATA OF ABSORPTION AND STRENGTH TESTS OF CONCRETE STAVES

Lot No	Dimensions of Staves, in.			Transverse Strength at 28 Days, lb. per in. of Width.			Compressive Strength, lb. per sq. in.			Absorption after 48 Hours, per cent.		
	Length.	Width.	Thickness.	High.	Low.	Average of Lot.	High.	Low.	Average of Lot.	High.	Low.	Average of Lot.
7142	30	10	2.55	128	98	111.0	3024	1870	2410	2.8	2.3	2.6
7125	36	11.7	2.70	98	71	89.0	2320	1650	2050	5.2	3.1	4.4
7112	30	12	1.85	58.5	54.0	57.0	1090	570	800	6.9	5.7	6.4
7080	30	9.9	2.6	57	42	49.0	2980	2230	2540	7.3	3.3	4.9
7109	30	9.7	2.5	83	70	76.0	2260	1650	1900	4.4	3.3	3.7
7107	30	10 0	2.5	154	115	128 0	4350	3040	3620	3 2	2.4	2.8
7123	30	10 05	2.5	100	80	92.0	2800	1220	1680	4.9	3.7	4.2

quirements for transverse strength will have sufficient compressive strength to sustain the loads imposed upon them. In view of this fact the committee decided not to include a compressive strength requirement in the specifications. There is also some difficulty in making a compressive strength test on account of the size and shape of the stave and the test on a section cut out from the stave may not always be representative of its quality. In preparing the following specifications the committee realized the advantage of requiring strength tests that could be made in the average plant. Such tests are of inestimable value in regulating manufacturing processes to improve the quality of products or to lower production costs or both.

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Standard Specifications and Building Regulations for Concrete Staves.*

Submitted, 1924, by Committee P-4, on Concrete Staves.

General

1. Concrete staves meeting the requirements of the following specifications may be used in the construction of silos, coal pockets, corn cribs, grain bins and other structures for which these units are suitable.

Tests.

2. Concrete staves must be subjected to transverse and absorption tests. All official tests must be made in a testing laboratory of recognized standing. Six samples representing the ordinary commercial product selected at random from stock must be provided for the purpose of testing.

Transverse Strength.

3. The ultimate transverse load on the test staves at 28 days must average not less than 90 lb. with no test falling below 75 lb. for each inch of width of the stave.

Absorption.

4. The absorption at 28 days must not in any case exceed 6 per cent.

Transverse Test.

5. The transverse test shall be made as follows: The sample to be tested shall be placed flatwise in the testing machine and supported at one end on a ⅝-in. round rod and at the other end by a spherical bearing block using a steel plate two inches wide and of sufficient stiffness to properly distribute the load between concrete and load point. The distance between points of support shall be exactly 24 inches. The load shall be applied at mid-span through a spherical bearing block.

6. In the absorption tests the samples shall be first thoroughly dried to a constant weight at a temperature not to exceed 230 deg. Fahrenheit. After drying, the sample shall be completely submerged in clean water at a temperature of between 60 and 80 deg. Fahrenheit for a period of 48 hours. The specimen shall then be removed, the surface water wiped off, and the sample reweighed. The percentage absorption is the weight of the water absorbed divided by the weight of the dry specimen and the quotient multiplied by 100.

Vertical Loading.

9. The load on any concrete stave wall, including the superimposed weight of the wall, shall not exceed 200 lb. per square inch.

Lateral Loading.

10. Silos, grain tanks, coal pockets, corn cribs, etc., constructed of concrete staves shall be hooped with steel rods or bands of such cross sectional area at the root of the thread in case of rods with cut threads that the steel will not be stressed to exceed 16,000 lb. per sq. in., and the hoops shall be placed at such intervals that the staves will not be loaded to exceed 25 per cent of their average transverse strength.

*Adopted as Tentative Standard, Annual Convention, February 26, 1924.

REPORT OF COMMITTEE P-1, ON STANDARD BUILDING UNITS.

Your Committee P-1, Standard Building Units, herewith submits its report.

Proposed Standard Specifications for Concrete Building Block and Concrete Building Tile.—Your committee has very carefully considered these specifications and also the suggestions made by various persons interested in the subject. As the result of the committee's deliberations, we recommend that the use of letters "A-B-C-D" to designate the various classifications of block and tile, be eliminated. Also that the former Class C light load-bearing block or tile be eliminated from the specifications and only the following classifications by word be considered, namely, "heavy load-bearing," "load-bearing" and "non-load-bearing" block or tile.

The revised draft of the specifications is attached hereto as exhibit A:

We recommend that the revised specifications for block and tile be continued as a tentative standard specification for another year.*

Proposed Standard Specifications of Concrete Brick.—Your committee desires to report that the unit weight of concrete, 140 lb. per cu. ft., section 5 under the sub-heading of "Absorption Requirements," be changed from 140 lb. per cu. ft. to 125 lb. per cubic foot. This recommendation is made due to the fact that a study of the weights of many specimens of concrete brick shows that the unit weight of concrete in brick is lower than the unit weight of concrete in block or tile. When this committee made the recommendation of 140 lb., it had no data on which to base its conclusions for brick and the revised figure has been determined by actual tests of the weight of concrete brick. This committee, therefore, recommends that the proposed standard specifications for concrete brick, with this change, be continued as tentative for another year.†

A revised draft of specifications is shown herewith as exhibit B:

Standardized Sizes.—A sub-committee was appointed consisting of H. A. Davis, Newton D. Benson, C. E. Lindsley, J. A. Ferguson, W. W. Plumb, J. H. McClatchy and J. W. Ochmann, which committee in joint meeting held June 21, 1923, at Washington, D. C., under the auspices of the Department of Commerce and Chamber of Commerce of the United States of America, considered the matter of standardization sizes very carefully and as a result of the sub-committee's work and this committee's deliberations, we recommend that sizes of block, tile and brick be:*

---

*The specifications were accepted as tentative, Feb. 26, 1924.

†The specifications were continued as tentative, Feb. 26, 1924.

Standard Sizes for Concrete Block:

Height	Width	Length
7⅝ in.	6 in.	15⅝ in.
7⅝ "	8 "	15⅝ "
7⅝ "	10 "	15⅝ "
7⅝ "	12 "	15⅝ "

Standard Sizes for Concrete Building Tile:

Height	Width	Length*
5 in.	3¾ in.	12 in.
5 "	8 "	12 "
5 "	12 "	12 "

Standard Sizes for Concrete Brick:

	ROUGH	
Height	Width	Length
2¼ in.	3¾ in.	8 in.
	SMOOTH	
2¼ in.	3⅞ in.	8 in.

The height, width and length of these block are taken on the block, tile or brick as they are laid in a wall.

We wish to emphasize that these are net dimensions, not nominal dimensions. It will be observed that the joint allowance in block is ⅜ in.

Forms for Reporting Tests for Concrete Building Units.—A sub-committee consisting of Stanton Walker was appointed to draft a suggested standard form for reporting tests. The sub-committee presents a suggested form.

This committee wishes to thank all members of the Institute and of the industry in general who have so kindly volunteered information or suggestions for the consideration of the committee, and invites the further co-operation of all who are interested in promoting the manufacture, sale and use of concrete building units.

W. R. HARRIS, *Chairman.*

*These sizes were approved by the convention, Feb. 26, 1924.

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Standard Specifications for Concrete Building Block and Concrete Building Tile.*

Submitted by Committee P-1, on Standard Building Units.

I. GENERAL.

1. The purpose of these specifications is to define the requirements for concrete building block and concrete building tile to be used in construction.

2. The word "concrete" shall be understood to mean portland cement concrete.

Strength Requirements.

3. According to the strength in compression 28 days after being manufactured or when shipped, concrete block and concrete tile shall be classified as heavy load bearing, load bearing, and non-load bearing on the basis of the following requirements.

	Compressive strength, lb. per sq. in. of gross cross-sectional area as laid in the wall.	
Name of Classification	Aver. of 3 or more Units	Min. for Individual Unit
Heavy load bearing block or tile......	1200	1000
Medium " " " " "......	700	600
Non- " " " " "......	250	200

4. The gross cross-sectional area of a one-piece concrete block or tile shall be considered as the product of the length times the width of the unit as laid in the wall. No allowance shall be made for air spaces in hollow units. The gross cross-sectional area of each unit of a two-piece block or tile shall be considered the product of the length of the unit times one-half the thickness of the wall for which the two-piece block or tile is intended.

5. The compressive strength of the concrete in units of all classifications except "non-load bearing block" shall be at least 1000 lb. per sq. in., when calculated on the minimum cross-sectional area in bearing.

Absorption Requirements

6. Concrete building block and tile to be exposed to soil or weather in the finished work (without stucco, plaster or other suitable protective covering) shall meet the requirements of the absorption test.

7. All concrete building block and tile not covered by paragraph 6 need not meet an absorption requirement.

---

* Adopted as Tentative Standard, Annual Convention, Feb. 26, 1924.

8. Concrete block and tile shall not absorb more than 10 per cent of the dry weight of the unit when tested as hereinafter specified, except when it is made of concrete weighing less than 140 lbs. per cu. ft. For block or tile made with concrete weighing less than 140 lbs. per cu. ft., the absorption in per cent by weight shall not be more than 10 multiplied by 140 and divided by the unit weight in pounds per cubic foot of the concrete under consideration.

Sampling.

9. Specimens for tests shall be representative of the commercial product of the plant.

10. Not less than three and preferably five specimens shall be required for each test.

11. The specimens used in the absorption test may be used for the strength test provided they have been dried at approximately 70° F. for not less than three days.

II. METHODS OF TESTING.

Strength Testing.

12. The specimens to be tested shall be carefully measured for overall dimensions of length, width and height.

13. Bearing surfaces shall be made plane by capping with plaster of paris or a mixture or portland cement and plaster which shall be allowed to thoroly harden before the test;

14. Specimens shall be accurately centered in the testing machine;

15. The load shall be applied through a spherical bearing block placed on top of the specimen;

16. When testing other than rectangular block or tile care must be taken to see that the load is applied through the center of gravity of the specimen;

17. Metal plates of sufficient thickness to prevent appreciable bending shall be placed between the spherical bearing block and the specimen;

18. The specimen shall be loaded to failure;

19. The compressive strength in pounds per square inch of gross cross-sectional area is the total applied load in pounds divided by the gross cross-sectional area in square inches.

sorption Test.

20. The specimens shall be dried to constant weight at a temperature of from 212 to 250° F. and the weight recorded. After drying, the specimen shall be immersed in clean water at approximately 70° F. for a period of 24 hours. They shall then be removed, the surface water wiped off and and the specimens re-washed. The absorption is the weight of the water absorbed, divided by the weight of the dry specimen and multiplied by one hundred.

Weight of Concrete.

21. The weight per cubic foot of the concrete in a block or tile is the weight of the unit in pounds divided by its volume in cubic feet. To obtain the volume of the unit, fill a vessel with enough water to immerse the specimen. The greatest accuracy will be obtained with the smallest vessel in which the specimen can be immersed with its length vertical. Mark the level of the water, then immerse the saturated specimen and weigh the vessel. Draw the water down to its original level and weigh the vessel again. The difference between the two weights divided by 62.5 equals the volume of the specimen in cubic feet.

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Standard Specifications for Concrete Brick.*

Submitted by Committee P-1, on Standard Building Units.

I. GENERAL.

Strength Requirements

1. The purpose of these specifications is to define the requirements for concrete brick to be used in construction.

2. The word "concrete" shall be understood to mean portland cement concrete.

3. The average compressive strength of concrete brick 28 days after being manufactured or when shipped shall not be less than 1,500 lb. per sq. in. of gross cross-sectioned area as laid in the wall, and the compressive strength of any individual brick shall not be less than 1,000 lb. per sq. in. of gross cross-sectional area as laid in the wall.

4. The gross cross-sectional area of a brick shall be considered as the product of the length times the width of the unit as laid in the wall.

Absorption Requirements

5. Concrete brick shall not absorb more than 12 per cent of the dry weight of the brick when tested as hereinafter specified except when they are made of concrete weighing less than 125 lb. per cu. ft. For brick made of concrete weighing less than 125 lb. per cu. ft., the average absorption in per cent by weight shall not be more than 12 multiplied by 125 and divided by the unit weight in pounds per cubic foot of the concrete under consideration.

Sampling

6. Specimens for tests shall be representative of the commercial product of the plant.

7. Five specimens shall be required for each test.

8. The specimens used in the absorption test may be used for the strength test provided they have been dried at approximately 70° F. for not less than three days.

II. METHODS OF TESTING.

Strength Test.

9. The specimens to be tested shall be carefully measured for overall dimensions of length, width and thickness.

10. Bearing surfaces shall be made plane by capping with plaster of paris or a mixture of portland cement and plaster which shall be allowed to thoroughly harden before the test;

11. Specimens shall be accurately centered in the testing machine;

12. The load shall be applied through a spherical bearing block placed on top of the specimen;

* Adopted as Tentative Standard, Annual Convention, Feb. 26, 1924.

13. Metal plates of sufficient thickness to prevent appreciable bending shall be placed between the spherical bearing block and the specimen;

14. The specimen shall be loaded to failure;

15. The compressive strength in pounds per square inch of gross cross-sectional area is the total applied load in pounds divided by the gross cross-sectional area in square inches.

16. The specimens shall be dried to constant weight at a temperature of from 212° to 250° F. and the weight recorded. After drying, the specimens shall be immersed in clean water at approximately 70° F. for a period of 24 hours. They shall then be removed, the surface water wiped off and the specimens re-weighed. The absorption is the weight of the water absorbed, divided by the weight of the dry specimen and multiplied by one hundred. Absorption Test

(Name of Testing Laboratory)

TESTS OF CONCRETE BUILDING UNITS

Manufacturer:..

Client:..

Type of Unit:..

Tests made in accordance with Proposed Standard Specifications for Concrete Building Block and Concrete Building Tile, and Proposed Standard Specifications for Concrete Brick, of the American Concrete Institute, 1924. See reverse of this sheet.

ABSORPTION TESTS

Mark: Lab.	Mark: Client	Date Tested	Dry Weight, Pound	Absorption, Per Cent by Wt.	Unit Weight of Concrete, lb. per cu. ft.	Corrected, Absorption Per cent*
Average						

* This information need not be furnished if the absorption by weight is within the required limits.

STRENGTH TESTS

Mark		Date Tested	Dimensions of Unit—in.			Loaded Area, sq. in.		Crushing Load		
								Total Pounds	Lb. per sq. in.	
Lab.	Client		Depth	Width	Length	Gross	Minimum		Gross Area	Minimum Area
Average										

Remarks:

Correct........................

Date of Report.................... Report No...........

Detach here if following information is confidential.

Report No...........

INFORMATION FURNISHED BY CLIENT

The following information is essential if recommendations are to be made for changes in method of manufacture.

DATA OF CONCRETE

Date Made	Types of Machinery: Mixer	Types of Machinery: Molding	Time of Mixing	Mixing Water per Sack of Cement	Units per Sack of Cement	Proportions of Cement to Fine and Coarse Aggregate	Time and Method of Curing

DATA OF AGGREGATES

Type of Aggregate	Sieve Anlayses* Amounts Coarser than each Sieve, Per cent by Weight								Fineness Modulus*	Unit Weight, Lb. per cu. ft.		Per cent Moisture by Weight as used
	100	48	28	14	8	4	3/8	3/4		Dry and Rodded	As used	

* This information will be furnished by the Laboratory at no additional charge, if samples of aggregate are submitted.

RECOMMENDATIONS

(Over)

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Standard Specifications for Plain Concrete Sewer Pipe.*

Submitted by Committee P-7, on Concrete Sewer Pipe.

I. General.

1. These specifications apply to concrete pipe intended to be used for the conveyance of sewage, industrial wastes and storm water. Pipe furnished under these specifications shall be of a single class. **Scope.**

2. The acceptability of pipe shall be determined by the results of the chemical and physical tests hereinafter specified, any or all of which may be required by the purchaser; and by inspection, to determine whether the pipe comply with the specifications as to dimensions, shape and freedom from external and internal defects. **Acceptability of Pipe.**

II. Materials.

3. Concrete sewer pipe shall be manufactured from portland cement concrete. By concrete is meant a suitable mixture of portland cement, mineral aggregates and water, hardened by hydraulic chemical action. **Concrete.**

4. Portland cement shall meet the requirements of the Standard Specifications and Tests for Portland Cement (Serial Designation C 9-21) of the American Society for Testing Materials. **Cement.**

5. The materials shall possess such physical and chemical properties that when molded into pipe and properly cured the product will be strong, durable and serviceable, free from objectionable defects, and in compliance with these specifications and tests. **Material in General.**

III. Sizes and Dimensions.

6. Pipe of the various sizes shall have the dimensions given in Table III for the quality of concrete therein designated. When concrete of a less unit strength is used, the dimensions of the pipe shall be such that the pipe will meet the test requirements given in Table II. **Dimensions.**

7. Permissible variations as given in Table IV shall in so case be exceeded. **Variations.**

---

* Adopted as Tentative Standard, Annual Convention, Feb. 25, 1924.

TABLE I.—DIMENSIONS, ETC., OF PLAIN CONCRETE SEWER PIPE FURNISHED AS TEST SPECIMENS.

Specimen No.	Date.	Job No.	Material.	Manufacturer.	Location of Factory.	Nominal Size, in.	Weight, lb.	Length, ft.	Thickness of Spigot, in.				Thickness of Socket, in.				Internal Diameter, in.						Remarks.
																	Spigot.		Back of Socket.		Socket.		
									Invert.	Crown.	Side.	Side.	Invert.	Crown.	Side.	Side.	Vertical.	Horizontal.	Vertical.	Horizontal.	Vertical.	Horizontal.	

TABLE II.—PHYSICAL TEST REQUIREMENTS OF PLAIN CONCRETE SEWER PIPE.

Internal Diameter, in.	Minimum Crushing Strength, lb. per lin. ft.		Maximum Absorption, per cent.
	When Tested by Sand Bearing Method.	When Tested by Three-edge Method.	
4	1430	950	8
6	1430	950	8
8	1430	950	8
10	1570	1050	8
12	1710	1140	8
15	1960	1310	8
18	2200	1470	8
21	2490	1660	8
24	3070	2050	8
27	3370	2240	8
30	3690	2460	8
33	4050	2700	8
36	4400	2930	8
39	4710	3140	8
42	5030	3350	8

NOTE.—The load per foot of pipe shall be determined by dividing the total test load by the laying length of the pipe in feet.

TABLE III.—DIMENSIONS OF PLAIN CONCRETE SEWER PIPE BASED ON CONCRETE HAVING A COMPRESSIVE STRENGTH OF NOT LESS THAN 4500 LB. PER SQ. IN. AT 28 DAYS.

Internal Diameter (D), in.	Laying Length (L), ft.	Inside Diameter at Mouth of Socket (Ds), in.	Depth of Socket (Ls), in.	Minimum Taper of Socket (H).	Thickness of Barrel (T), in.	Thickness of Socket (Ts).
4	2, $2\frac{1}{2}$	6	$1\frac{1}{2}$	1:20	$\frac{9}{16}$	The thickness of the socket $\frac{1}{4}$ in. from its outer end shall be not less than three-fourths of the thickness of the barrel of the pipe.
6	2, $2\frac{1}{2}$	$8\frac{1}{4}$	2	1:20	$\frac{5}{8}$	
8	2, $2\frac{1}{2}$, 3	$10\frac{3}{4}$	$2\frac{1}{4}$	1:20	$\frac{3}{4}$	
10	2, $2\frac{1}{2}$, 3	13	$2\frac{1}{2}$	1:20	$\frac{7}{8}$	
12	2, $2\frac{1}{2}$, 3	$15\frac{1}{4}$	$2\frac{1}{2}$	1:20	1	
15	2, $2\frac{1}{2}$, 3	$18\frac{3}{4}$	$2\frac{1}{2}$	1:20	$1\frac{1}{4}$	
18	2, $2\frac{1}{2}$, 3	$22\frac{1}{4}$	$2\frac{3}{4}$	1:20	$1\frac{1}{2}$	
21	2, $2\frac{1}{2}$, 3	26	$2\frac{3}{4}$	1:20	$1\frac{3}{4}$	
24	2, $2\frac{1}{2}$, 3	$29\frac{1}{2}$	3	1:20	$2\frac{1}{8}$	
27	3	$33\frac{1}{4}$	$3\frac{1}{4}$	1:20	$2\frac{1}{4}$	
30	3	37	$3\frac{1}{2}$	1:20	$2\frac{1}{2}$	
33	3	$40\frac{1}{4}$	4	1:20	$2\frac{3}{4}$	
36	3	44	4	1:20	3	
39	3	$47\frac{1}{4}$	4	1:20	$3\frac{1}{4}$	
42	3	51	4	1:20	$3\frac{1}{2}$	

NOTE.—When pipes are furnished having an increase in thickness over that given in the last column, then the diameter of socket shall be increased by an amount equal to twice the increase of thickness of barrel

IV. WORKMANSHIP AND FINISH.

Finish. 8. Pipe shall be substantially free from fractures, large or deep cracks and surface roughness.

Pipe Ends. 9. The ends of the pipe shall be square with their longitudinal axes.

Specials. 10. Special shapes shall have a plain spigot end and a socket end corresponding in all respects with the dimensions specified for pipe of the corresponding internal diameter. Branches shall be furnished to lay the same length as straight pipe. All specials shall conform in finish to the requirements for straight pipe.

Slants. 11. Slants shall have their spigot ends cut at an angle of approximately 45 degrees with the longitudinal axis.

Curves. 12. Curves shall cover arcs of 90, 45, 22½ deg., as required. They shall conform substantially to the curvature as specified.

Branches. 13. Branches shall be furnished with the connection or connections of the size or sizes specified, securely and completely fastened in the process of manufacture to the barrel of the pipe. T-branches and double T-branches shall have the branch axis perpendicular to the longitudinal axis of the pipe. Y-branches, double Y-branches, and V-branches shall have their branch axes approximately 45 degrees from the longitudinal axis of the pipe measured from the socket end. All branches shall terminate in sockets, and the barrel of the branch shall be of sufficient length to permit making a proper joint when the connecting pipe is inserted in the branch socket.

Channels. 14. Channel, or split pipe, shall be accurate half sections of the corresponding size of pipe.

V. MARKING.

Marking. 15. When shipment of pipe is made in any manner other than direct from manufacturer to user, pipe shall be so marked that the manufacturer of the pipe can be identified.

VI. CHEMICAL TESTS AND REQUIREMENTS.

Chemical Test. 16. The purchaser may specify special chemical requirements as to resistance of the pipe to chemical action in cases where soils, industrial wastes, sewage, or ground waters have marked acid or alkaline character, or are of abnormally high temperature, and may prescribe chemical tests of the pipe to ascertain whether these special requirements are met. Without a special agreement in advance, no pipe shall be rejected by reason of its composition as determined by ultimate chemical analysis.

TABLE IV.—PERMISSIBLE VARIATIONS IN DIMENSIONS OF PLAIN CONCRETE SEWER PIPE.

Normal Size, inches.	Limits of Permissible Variation in:				
	Length, inches per foot (−).	Internal Diameter, in.		Depth of Socket, inches (−).	Thickness of Barrel, inches (−).
		Spigot (±).	Socket (±).		
4	$\frac{1}{4}$	$\frac{1}{8}$	$\frac{1}{8}$	$\frac{1}{8}$	$\frac{1}{16}$
6	$\frac{1}{4}$	$\frac{3}{16}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{1}{16}$
8	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{16}$
10	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{16}$
12	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{16}$
15	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{3}{32}$
18	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{3}{32}$
21	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{1}{8}$
24	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{1}{8}$
27	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{1}{8}$
30	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{8}$
33	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{3}{8}$
36	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{3}{16}$
39	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{3}{16}$
42	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{3}{16}$

NOTE.—The minus sign (−) alone indicates that the plus variation is not limited; the plus and minus sign (±) indicates variation in both excess and deficiency in dimension.

VII. PHYSICAL TESTS.

(A) *General.*

Tests Required. 17. The physical tests of pipe shall include: Load test, hydrostatic pressure test and absorption test.

Selection of Specimens. 18. The specimens to be tested shall be selected by the purchaser or his representative at the point or points designated by him when placing the order. They may be selected in such numbers as are judged necessary to determine fairly the quality of all the pipe. The manufacturer or seller shall furnish specimens for test, without charge, up to one-half of 1 per cent of the whole number of pipe ordered in each size, and the purchaser shall pay for all in excess of that percentage at the same rate as for other pipe.

Quality of Specimens. 19. The test specimens shall be full size pipe which will in every respect pass the inspection requirements hereinafter provided. They shall be free from all visible moisture, measured and weighed and the results recorded and preserved as shown in Table I.

Complying with Test. 20. Failure of 20 per cent of the specimens to meet the requirements of any of the tests imposed, shall result in rejection of all the pipe in the shipment or delivery, corresponding to the sizes thus failing to comply;

except that in the event of 20 per cent of the specimens in any size failing to meet the requirements, the manufacturer or seller may, with the consent of the consumer or purchaser, furnish for test, without charge, additional specimens from the same shipment. In case more than 80 per cent of the specimens tested, including those first tested, shall show substantial compliance for each of the various tests performed, then the entire shipment or delivery for this size shall be accepted, otherwise it shall be rejected.

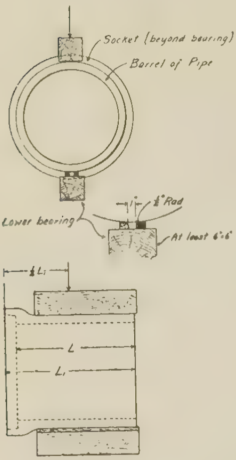


FIG. 1.—THREE-EDGE BEARINGS.

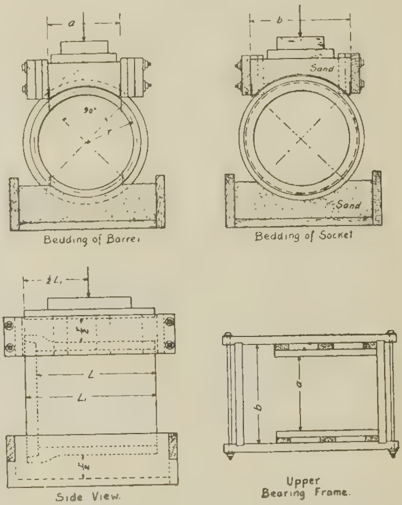


FIG. 2.—SAND BEARINGS.

(B) *Load Test.*

Test Methods.

21. The load test shall be performed either by the sand bearing or three-edge bearing method as hereinafter described. Each manufacturer shall be fully equipped to test by at least one of these methods.

Sand Bearing Test.

22. When sand bearings are used, the specimen shall have its upper and lower quadrants measured on the center line of the shell, carefully bedded in dry, clean, uniformly compacted sand having a minimum thickness of at least one-fourth the mean diameter of the pipe to be tested. The sand shall be prevented from lateral flow by being placed in rigid frames which shall not come in contact with the test specimen. Seepage of sand between the pipe and the upper bearing frame may be prevented by strips

of cloth attached to the lower inside edge of the frame. The upper bearing shall be centered over the pipe and the load shall be applied centrally thereto through a rigid horizontal bearing plate, covering its surface but not in contact with the bearing frame.

23. The sand bearing test may be made without the use of a testing machine, by piling weights directly on a platform resting on, and fully supported by, the top bearing plate, provided, however, that the weight shall be piled symmetrically about a vertical line through the center of the pipe, and that the platform shall not be allowed to touch the top bearing frame. **Test by Weights.**

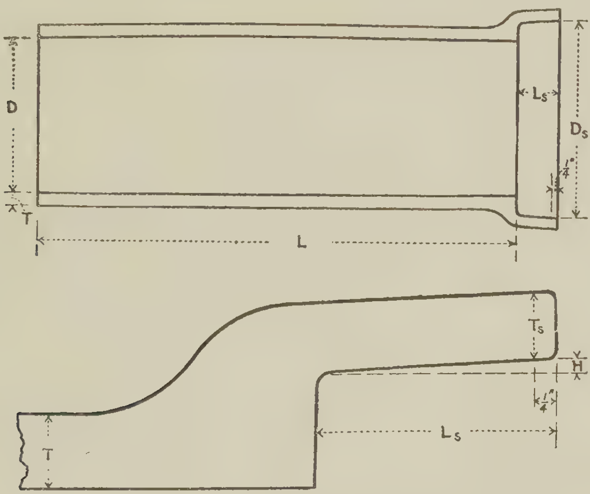


FIG. 3.—APPLICATION OF DIMENSIONS GIVEN IN TABLE III.

24. When three-edge bearings are used, the ends of each specimen of pipe shall be accurately marked in halves of the circumference prior to the test. The lower bearings shall consist of two wooden strips with vertical sides, each strip having its interior top corner rounded to a radius of approximately ½ inch. They shall be straight, and shall be securely fastened to a rigid block with their interior vertical sides 1 inch apart. The upper bearing shall be a wooden block, straight and true from end to end. The bearings shall be centered on the diametrically opposite markings previously made and the test load shall be applied through the upper bearing block in such a way as to produce a uniform distribution of the load throughout the length of the pipe and leave the bearing free to move in a vertical plane passing midway between the lower bearings. **Three-edge Bearing Test.**

25. In testing a pipe which is "out of line" the lines of the bearings chosen shall be those which appear to give most favorable conditions for fair bearings. **Pipe "out of line."**

Application of Load.

(Machine Testing).

26. Any prime mover or hand power which will apply the load at a uniform rate of about 2,000 lb. per minute, or in increments of not more than 100 lb. at the same rate, may be used in making the test. The testing machine shall be substantial and rigid throughout, so that the distribution of the load will not be affected appreciably by the deformation or yielding of any part. The load shall be applied continuously until failure occurs. The load at the time of failure shall be observed and recorded.

(C) *Hydrostatic Test.*

27. Sound full size pipe shall be tested for leakage under internal hydrostatic pressure.

Application of Internal Pressure.

28. The ends of the pipe shall be closed by watertight bulkheads, and internal water pressure, as measured by a standardized gauge, applied to the specimen as follows:

5	lb.	per	square	inch	for	5	minutes
10	"	"	"	"	"	10	"
15	"	"	"	"	"	15	"

Test Result.

29. The specimens shall be capable of withstanding these pressures and shall show no active leakage during the final 5 minutes of the 15 lb. test period. Moisture appearing on the surface of the pipe in the form of patches or beads, adhering to the surface, shall not be considered leakage.

(D) *Absorption Test.*

Absorption Test.

30. The specimens shall be sound pieces with all edges broken, and may be from pipe broken in the crushing or other tests. They shall be from 12 to 20 sq. in. in area, and shall be as nearly square as they can be readily prepared.

Specimen Markings.

31. Each specimen shall be marked so that it may be identified with the pipe from which it was taken. The marking shall be applied so that the pigment used shall not cover more than 1 per cent of the total superficial area of the specimen.

Drying Specimens.

32. Preparatory to the absorption test, the specimen shall be first weighed and then dried in a drier or oven at a temperature of not less than 230 deg. F. (110 deg. C.) to constant weight.

Balance Used.

33. The balance used shall be sensitive to 0.5 g. when loaded with 1 kg. and weighings shall be read to the nearest gram. When other than metric weights are used, the same degree of accuracy shall be obtained.

Boiling Specimens.

34. The specimen, after drying, cooling and weighing, shall be placed in a suitable receptacle, covered with distilled water or rain water, and boiled for five hours, and then cooled in water to a final temperature of 50 to 59 deg. F. (10 to 15 deg. C.).

35. The specimen shall be allowed to drain for not more than one minute, the superficial moisture removed by towel or blotting paper, and then weighed. **Final Weighing.**

36. The absorption shall be considered as the increase in weight over the dry weight and shall be calculated and recorded as a percentage thereof. The results shall be reported for each specimen together with the average for each size of pipe represented. **Absorption Results.**

(E) *Physical Test Requirements.*

37. Concrete pipe shall meet the load and absorption test requirements as given in Table II.

VIII. INSPECTION.

38. All pipe shall be subjected to inspection at the factory, trench or other point of delivery by a competent inspector employed by the consumer or purchaser. The purposes of the inspection shall be to cull and reject pipe which, independent of the physical tests herein specified, fail to meet the requirements of these specifications.* **Purpose of Inspection.**

39. Pipe shall be subject to rejection on account of any of the following: **Causes for Rejection.**

(a) Variations in any dimension exceeding the permissible variations given in Table IV.

(b) Fracture or cracks passing through the shell or socket, except that a single crack at either end of a pipe not exceeding 2 in. in length or a single fracture in the socket not exceeding 3 in. in width nor 2 in. in depth will not be deemed cause for rejection unless these defects exist in more than 5 per cent of the entire shipment or delivery.

(c) Defects which indicate imperfect mixing and molding.

(d) Variations of more than ⅛ in. per linear foot in alignment of a pipe intended to be straight.

(e) Failure to give a clear ringing sound when a dry pipe placed on end is tapped with a light hammer.

(f) Insecure attachment of branches or spurs.

40. All rejected pipe shall be plainly marked by the inspector and shall be replaced by the manufacturer or seller with pipe which meet the requirements of these specifications, without additional cost to the consumer or purchaser. **Rejected Pipe.**

---

* When the point of inspection is not designated in the contract or order, it shall be understood as being at the point of manufacture.

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Standard Specifications for Concrete Drain Tile.*

Submitted by Committee P-7, on Concrete Sewer Pipe.

I. General.

Scope.

1. These specifications apply to concrete drain tile, consisting of two classes, namely, Standard Drain Tile and Extra-Quality Drain Tile.

The purposes for which these classes are intended to be suitable are as follows:

Classes.

Standard Drain Tile, for ordinary district land drainage at moderate depths.

Extra-Quality Drain Tile, for district land drainage, at considerable depths and where an extra quality is desired.

Basis of Purchase.

2. The purchaser shall specify the class of tile to be supplied. Standard Drain Tile shall be supplied where no advance selection is stated.

Acceptability of Tile.

3. The acceptability of drain tile shall be determined by the results of the chemical and physical tests hereinafter specified, any or all of which may be required by the purchaser; and by inspection, to determine whether the tile comply with the specifications as to dimensions, shape and freedom from external and internal defects.

II. Material.

Concrete.

4. Concrete drain tile shall be manufactured from portland cement concrete. By concrete is meant a suitable mixture of portland cement, mineral aggregates and water, hardened by hydraulic chemical action.

Cement.

5. Portland cement shall meet the requirements of the Standard Specifications and Test for Portland Cement (Serial Designation: C 9-21) of the American Society for Testing Materials.

Material in General.

6. The materials shall possess such physical and chemical properties that when molded into tile and properly cured the product will be strong, durable and serviceable, free from objectionable defects, and in compliance with these specifications and tests.

---

* Adopted as Tentative Standard, Annual Convention, Feb. 25, 1924.

III. SIZES AND DIMENSIONS.

7. All drain tile shall be of circular cross-section, except when otherwise specified in advance. They shall be approximately straight, except in the case of special connections. The ends shall be so regular and smooth as readily to admit of making close joints by pressing together adjoining tile. **Shape of Tile.**

8. The sizes of drain tile shall be designated by their internal diameters. **Sizes.**

9. Drain tile smaller than 12 in. in diameter shall have a minimum length of 12 in. Tile from 12 to 30 in. in diameter, inclusive, shall have lengths not less than the diameters. Tile larger than 30 in. in diameter shall have a minimum length of 30 in. **Lengths.**

10. Permissible variations as given in Table I shall in no case be exceeded. **Permissible Variation.**

TABLE I.—PERMISSIBLE VARIATIONS IN DRAIN TILE.

Physical Properties Specified.	Standard Drain Tile.	Extra-Quality Drain Tile.
Allowable variation of average diameter of any tile above or below specified diameter, per cent................	3	3
Allowable variation from straightness, percentage of length..	2	2

IV. WORKMANSHIP AND FINISH.

11. Drain tile shall be substantially uniform in structure throughout and shall be reasonably smooth on the inside. **Finish.**

12. Drain tile shall be free from cracks and checks extending into the body of the tile in such a manner as to decrease the strength appreciably. Tile shall not be chipped or broken in such a manner as to decrease their strength materially or to admit earth into the drain. **Cracks.**

V. MARKING.

13. When shipment of tile is made in any manner other than direct from manufacturer to user, tile shall be so marked that the manufacturer of the tile can be identified. **Marking**

VI. CHEMICAL TEST AND REQUIREMENTS.

14. The purchaser may specify special chemical requirements as to resistance of the tile to chemical action in cases where soils or drainage waters have marked acid or alkaline character, or are of abnormally high **Chemical Tests and Requirements.**

temperature, and may prescribe chemical tests of the tile to ascertain whether these special requirements are met. Without a special agreement in advance, no drain tile shall be rejected by reason of its composition as determined by ultimate chemical analysis.

VII. PHYSICAL TESTS.

(A) *General.*

Tests Required.

15. The physical tests of drain tile shall include Load Tests and Absorption Test, and may include Freezing and Thawing Test, when specified by the purchaser in advance or when called for by the manufacturer or other seller as provided herein.

Selection of Specimens.

16. The specimens to be tested shall be selected by the purchaser or his representative at the point or points designated by him when placing the order. The test specimen shall be full size tile which will in every respect pass the inspection requirements hereinafter provided.

Number of Test Specimens.

17. A standard physical test shall comprise tests of five individual tile for each size of tile represented. Specimens of tile may be selected by the inspector in such number as he judges necessary to determine fairly the quality of all the tile. The manufacturer or other seller shall furnish specimens of tile without charge up to 0.5 per cent of the whole number of tile, and the purchaser shall pay for all in excess of that percentage at the same rate as for other tile.

(B) *Load Tests.*

Test Methods.

18. The load test shall be performed either by the sand bearing or three-edge bearing method as hereinafter described. Each manufacturer shall be fully equipped to test tile by at least one of these methods.

Preparation of Test.

19. The walls of the tile shall, at the time of testing, be as thoroughly wet as will result from completely covering with hay, cloth, or similar absorbent material, and keeping the covering wet for not less than 12 hours.

Temperature Conditions.

20. No specimen of tile shall be exposed to water or air temperatures lower than 40 deg. F. from the beginning of wetting until tested. Frozen tile shall be completely thawed before the wetting begins.

Weighing.

21. Each specimen of tile shall be weighed on reliable scales just prior to testing, and the weights shall be reported.

Sand Bearing Test.

22. When sand bearings are used, the specimen shall have its upper and lower quadrants, measured on the center line of the shell, carefully bedded in dry, clean, uniformly compacted sand having a minimum thickness of at least ¼ the mean diameter of the pipe to be tested. The sand

shall be prevented from lateral flow by being placed in rigid frames which shall not come in contact with the test specimen. Seepage of sand between the tile and the upper bearing frame may be prevented by strips of cloth attached to the lower inside edge of the frame.

The upper bearing shall be centered over the tile and the load shall be applied centrally thereto through a rigid horizontal bearing plate, covering its surface but not in contact with the bearing frame.

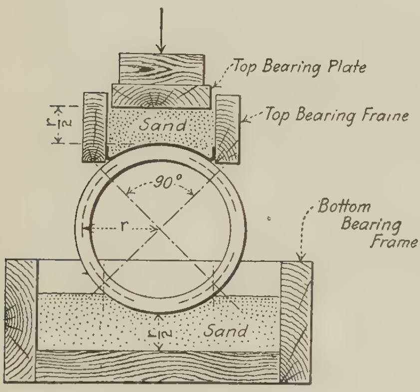


FIG. 1.—SAND BEARINGS.

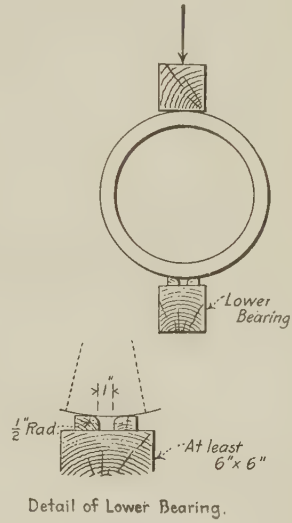


FIG. 2.—THREE-POINT BEARINGS.

23. The sand bearing test may be made without the use of a testing machine, by piling weights directly on a platform resting on, and fully supported by, the top bearing plate, provided, however, that the weight shall be piled symmetrically about a vertical line through the center of the tile, and that the platform shall not be allowed to touch the top bearing frame. **Test by Weights.**

24. When three-edge bearings are used, the ends of each specimen of tile shall be accurately marked in halves of the circumference prior to the test. The lower bearings shall consist of two wooden strips with vertical sides, each strip having its interior top corner rounded to a radius of approximately ½ inch. They shall be straight, and shall be securely fastened to a rigid block with their interior vertical sides 1 inch apart. The upper bearing shall be a wooden block, straight and true from end to end. The bearings shall be centered on the diametrically opposite markings previously made and the test load shall be applied through the upper bearing block in such a way as to produce a uniform distribution of the load throughout the length of the pipe and leave the bearing free to move in a vertical plane passing midway between the lower bearings. **Three-edge bearing Test.**

Tile "out of line."

25. In testing tile which is "out of line" the lines of the bearings chosen shall be those which appear to give most favorable conditions for fair bearings.

Application of Load.

Testing Machines.

26. Any prime mover or hand power which will apply the load at a uniform rate of about 2,000 lb. per minute, or in increments of not more than 100 lb. at the same rate, may be used in making the test. The testing machine shall be substantial and rigid throughout, so that the distribution of the load will not be affected appreciably by the deformation or yielding of any part. The load shall be applied continuously until failure occurs. The load at the time of failure shall be observed and recorded.

(C) *Absorption Test.*

Absorption Test Specimens.

27. Absorption test specimens may be taken from tile broken in the load test. Three specimens shall be taken from each of five tile,—one specimen from each end of the tile and the third specimen from near the middle portion. All specimens shall be sound pieces with all edges broken and shall be from 12 to 20 sq. in. in area, and as nearly square as they can readily be prepared.

Specimen Markings.

28. Each specimen shall be marked so that it may be identified with the tile from which it was taken. The markings shall be applied so that the pigment used will not cover more than 1 per cent of the total area of the specimen.

c .s.

29. Preparatory to the absorption test, all specimens shall be first weighed and then dried in a drier or oven, at a temperature of not less than 110 deg. C. (230 deg. F.) to constant weight.

Balance Used.

30. The balance used shall be sensitive to 0.5 g. when loaded with 1 kg., and weighings shall be read at least to the nearest gram. When other than metric weights are used, the same degree of accuracy must be obtained.

Boiling Specimen.

31. Specimens after drying and weighing shall be placed in a suitable receptacle, covered with distilled water or rainwater, and boiled for five hours, and then cooled in water to a final temperature of 10 to 15 deg. C. (50 to 59 deg. F.).

Final Weighing.

32. The specimens shall be allowed to drain for not more than one minute, superficial moisture removed by towel or blotting paper, and then weighed.

Absorption Results.

33. The absorption shall be considered as the increase in weight over the dry weight and shall be calculated and recorded as a percentage thereof. The results shall be reported for each specimen, together with the average for the fifteen or more specimens comprising the standard sample.

(D) *Freezing and Thawing Test.*

34. For the freezing and thawing test another set of specimens similar to those used in the absorption test shall be prepared and dried to constant weight. **Freezing and Thawing Specimen.**

35. The specimens shall be immersed for 72 hours in water having a temperature of 18 to 24 deg. C. (65 to 75 deg. F.), and then weighed. When the specimens have been weighed after saturation with water, they shall be returned to the water and kept immersed until the freezing test is begun. For freezing, they shall be placed with the concave faces upward in watertight metal trays, suitably mounted in a rigid metal crate, and immersed in ice water until the specimens have attained substantially the temperature of the water, after which the water shall be drawn to a depth of ½ in. in each tray. The crate shall then be lifted without disturbing the specimens, and placed in the freezing apparatus. **Preliminary Treatment.**

36. Freezing shall be performed in a quiet atmosphere, free from perceptible natural or artificial currents. If artificial freezing apparatus is employed, the apparatus shall have sufficient heat-absorbent capacity to enable the temperature of the freezing chamber to be brought to —10 deg. C. (14 deg. F.), or below, within thirty minutes after the introduction of the specimens. The temperature in the freezing apparatus shall not fall lower than —20 deg. C. (—4 deg. F.). The freezing shall be continued until the water in the trays is frozen solid. **Freezing.**

37. At the conclusion of freezing the crate of specimens shall be withdrawn and at once immersed in water at a temperature of 18 deg. to 24 deg. C. (65 deg. to 75 deg. F.) in a special receptacle of proper size. A temperature of 18 deg. to 24 deg. C. (65 deg. to 75 deg. F.) shall then be maintained for not less than 2 hours. At the conclusion of the thawing treatment, the crate of specimens shall be inspected and the condition of each sample shall be noted in the records. **Thawing.**

38. Failure under the freezing and thawing treatment shall be considered to be reached when: **Freezing and Thawing Results.**

(a) The specimens show superficial disintegration or spalling with loss of weight of more than 5 per cent of the initial dry weight, or

(b) The specimens show evident serious loss of structural strength.

(E) *Physical Test Rrequirements.*

39. The average test load for all of the tile constituting a standard sample shall not be less than given in Table II, and no individual tile shall show a strength of less than 75 per cent of such requirement. **Load Test Requirements.**

40. The average absorption for all the specimens constituting a standard sample shall not be greater than that given in Table II, and the average test of an individual tile shall in no case show an absorption of more than 125 per cent of such requirement. **Absorption Test Requirement.**

Freezing Test Requirement.

41. In the freezing and thawing test, at least 95 per cent of all the tile tested shall meet the requirements.

Alternative Freezing Test.

42. In the event that a standard sample of tile fails to meet the requirements of the absorption test, the manufacturer or other seller may demand recourse to the freezing and thawing test, to be made at his expense. In such recourse, the number of tiles tested shall be four times the number represented by the standard sample. If the material passes the freezing and thawing test satisfactorily, it shall not be rejected on account of its failure to meet the absorption requirements.

Re-test.

43. In the event of the failure of a standard sample to meet the above requirements, the manufacturer or other seller may thoroughly cull the material and submit a portion for re-test at his own expense, and for such re-test the number of tiles per sample shall be 10 for the strength and absorption tests and 20 for the freezing and thawing test. In the event of the failure of the material after culling to pass the requirements, it shall be rejected without further test.

TABLE II.—PHYSICAL TEST REQUIREMENTS.

Drain Tile.

Internal Diameter of Tile, inches.	Standard Drain Tile. Average Load, lb. per lin. ft. Sand Bearing.	Standard Drain Tile. Average Load, lb. per lin. ft. 3-edge Bearing.	Standard Drain Tile. Average Absorption, per cent.	Extra-quality Drain Tile. Average Load, lb. per lin. ft. Sand Bearing.	Extra-quality Drain Tile. Average Load, lb. per lin. ft. 3-edge Bearing.	Extra-quality Drain Tile. Average Absorption, per cent.
4	1200	800	10	1600	1067	9
6	1200	800	10	1600	1067	9
8	1200	800	10	1600	1067	9
10	1200	800	10	1600	1067	9
12	1200	800	10	1600	1067	9
14	1200	800	10	1600	1067	9
16	1300	867	10	1600	1067	9
18	1400	933	10	1800	1200	9
20	1500	1000	10	2000	1333	9
22	1600	1067	10	2200	1467	9
24	1700	1133	10	2400	1600	9
26	1800	1200	10	2600	1733	9
28	1900	1267	10	2800	1867	9
30	2000	1333	10	3000	2000	9
32	2100	1400	10	3200	2133	9
34	2200	1467	10	3400	2267	9
36	2300	1533	10	3600	2400	9
38	2400	1600	10	3800	2533	9
40	2500	1667	10	4000	2667	9
42	2600	1733	10	4200	2800	9

NOTE.—When the freezing and thawing test is made as provided herein, the number of freezings and thawings to be endured shall be as follows: for standard drain tile, 36; for extra-quality drain tile, 48.

VII. INSPECTION.

44. All tile shall be subject to inspection at the factory, trench or other point of delivery by a competent inspector employed by the consumer or purchaser. The purposes of the inspection shall be to cull and reject tile which, independent of the physical tests herein specified, fail to meet the requirements of these specifications.* **Purpose of Inspection.**

45. Tile shall be subject to rejection on account of any of the following: **Causes of Rejection.**

(a) Variations in any dimensions exceeding the permissible variations given in Table I.

(b) Cracks, fractures or defects in excess of those allowed in Sections 11 and 12.

(c) Failure to give a clear ringing sound when a dry tile placed on end is tapped with a light hammer.

46. All rejected tile shall be plainly marked by the inspector and shall be replaced by the manufacturer or seller with tile which meets the requirements of these specifications, without additional cost to the consumer or purchaser. **Rejected Tile.**

47. The manufacturer or other seller may appeal from decisions of the inspector on questions of strength or structure when such decisions are based on visual inspection alone, in which case the point at issue shall be determined by standard physical tests, the cost of which shall be paid by the appellant, if the inspector was right, or by the purchaser if his inspector was in error. **Appeal from Inspector.**

---

* When the point of inspection is not designated in the contract or order, it shall be understood as being at the point of manufacture.

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Standard Specifications for Reinforced-Concrete Sewer Pipe.*

Submitted by Committee P-7, on Concrete Sewer Pipe.

I. General.

Scope and Classes.

1. These specifications apply to reinforced concrete pipe intended to be used for the conveyance of sewage, industrial waste and storm water. Pipe furnished under these specifications shall be of two classes to be known respectively as Class A pipe and Class B pipe.

Acceptability of Pipe.

2. The acceptability of pipe shall be determined by the purchaser; by the results of the load tests hereinafter specified, if and when required, and by inspection, to determine whether the pipe comply with the specifications as to dimensions, shape, and freedom from external and internal defects.

II. Materials.

Concrete and Steel.

3. Pipe shall be manufactured from concrete in which steel has been imbedded in such manner that the steel and concrete shall assist each other in taking stress.

By concrete is meant a suitable mixture of portland cement, mineral aggregates and water, hardened by hydraulic chemical action.

Cement.

4. Portland cement shall meet the requirements of the Standard Specifications and Tests for Portland Cement (Serial Designation, C 9-21) of the American Society for Testing Materials.

Steel.

5. Reinforcement may consist of wire or fabric which will meet the requirements of the Tentative Specifications for Cold-Drawn Steel Wire for concrete reinforcement (Serial Designation: A 82-21 T) of the American Society for Testing Materials, or of rods and bars which meet the requirements of the Standard Specifications, for Billet Steel Concrete Reinforcement bars (Serial Designation A-15-15).

Material in General.

6. The materials shall possess such physical properties that when molded into pipe and properly cured, the product will be strong, durable and serviceable, free from objectionable defects, and in compliance with these specifications and tests.

Mixture.

7. The aggregate shall be so graded and proportioned and thoroughly mixed with such a proportion of cement and water as to produce a homogeneous concrete mix of such quality that the pipe will meet the test requirements and inspection hereinafter provided.

---

* Adopted as Tentative Standard, Annual Convention, Feb. 25, 1924.

III. DESIGN.

8. The shell thickness and amount of reinforcement shall be such that the pipe will meet the load test requirements herein provided. **Design.**

9. The reinforcement shall extend throughout the barrel of the pipe. It shall be assembled into units so designed that they may be readily placed and maintained of true, exact shape and proper position within the pipe form during the manufacturing process. **Reinforcement.**

10. The ends of reinforced concrete pipe shall be so formed that when the pipe are laid together and the joints cemented, they shall make a continuous and uniform line of pipe with a smooth and regular interior surface. The joints shall be of such a design that when cemented they will prevent leakage and infiltration as well as appreciable irregularities in the flow line of the sewer. **Joints.**

11. Reinforcement shall be placed not less than ¾ in. from the surface of the pipe shell. **Position of Reinforcement.**

IV. WORKMANSHIP AND FINISH.

12. Pipe shall be substantially free from fractures, large or deep cracks and surface roughness. The ends of pipe shall be square with their longitudinal axes. **Finish.**

13. Variations of the internal diameter shall not exceed 2 per cent. The shell thickness shall not be less than that intended in the design by more than 5 per cent at any point. **Variations.**

V. MARKING.

14. When shipment of pipe is made in any manner other than direct from manufacturer to user, pipe shall be so marked that the manufacturer of the pipe can be identified. The date of manufacture shall be plainly marked on the pipe in all cases. **Markings.**

VI. LOAD TEST.

15. The test specimens shall be full size pipe which will in every respect pass the inspection requirements hereinafter provided. **Quality of Specimen.**

16. The specimens to be tested shall be selected by the purchaser or his representative at the point or points designated by him when placing the order. The manufacturer shall furnish, for testing purposes and at his own expense, one pipe of each size included in the order; the purchaser bearing all expense of testing such pipe. Should additional tests be made upon the demand of the purchaser or manufacturer, as hereinafter provided, then cost of such additional test specimens and the expense of testing shall be borne by the party making such demand. **Selection of Specimen.**

Complying with Test.

17. Should the test specimens furnished by the manufacturer meet the test requirements, then all pipe represented by such specimens shall be accepted; provided, however, that the purchaser shall have the right to require an additional test in any size or sizes of pipe.

Should any of the specimens first tested fail to meet the test requirements, then the manufacturer shall have the right to submit an additional test of the size or sizes of pipe which have failed.

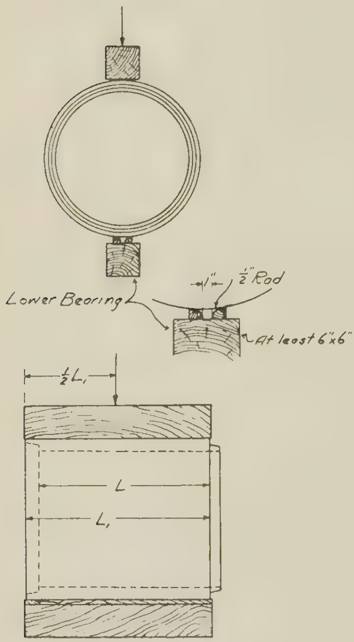


FIG. 1.—THREE-EDGE BEARINGS.

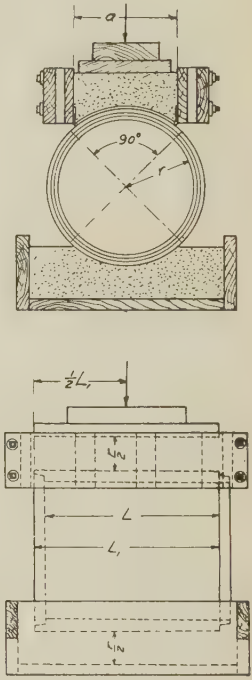


FIG. 2.—SAND BEARINGS.

In either case, when an additional test is made in any size of pipe, such test shall consist of three pipe or multiple thereof including the pipe first tested; if two-thirds of the pipe so tested shall meet the test requirements, then all pipe represented by such test shall be accepted, otherwise they shall be rejected.

Test Methods.

18. The load test shall be performed either by the sand bearing or three-edge bearing methods as hereinafter described.

Sand Bearing Test.

19. When sand bearings are used, the specimen shall have its upper and lower quadrants, measured on the center line of the shell, carefully bedded in dry, clean, uniformly compacted sand having a minimum thickness of at least one-quarter the mean diameter of the pipe to be tested. The sand shall be prevented from lateral flow by being placed in rigid frames which shall not come in contact with the test specimen. Seepage of sand between the pipe and the upper bearing frame may be prevented by strips of cloth attached to the lower inside edge of the frame.

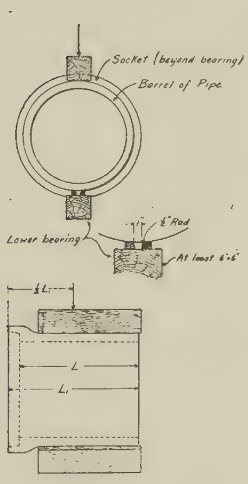


FIG. 3.—THREE-EDGE BEARINGS.

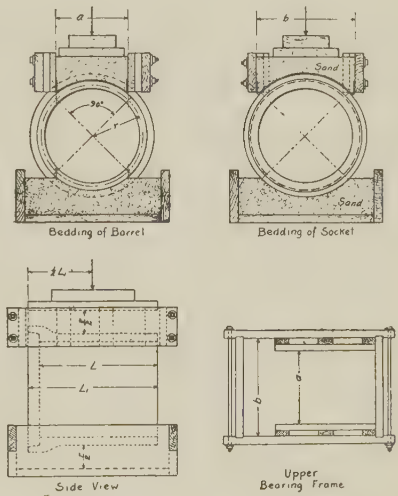


FIG. 4.—SAND BEARINGS.

The upper bearing shall be centered over the pipe and the load shall be applied centrally thereto through a rigid horizontal bearing plate, covering its surface but not in contact with the bearing frame.

Test by Weights.

20. The sand bearing test may be made without the use of a testing machine, by piling weights directly on a platform resting on, and fully supported by, the top bearing plate, provided, however, that the weights shall be placed symmetrically about a vertical line through the center of the pipe, and that the platform shall not be allowed to touch the top bearing frame.

Three-edge Bearing Test.

21. When three-edge bearings are used, the ends of each specimen of pipe shall be accurately marked in halves of the circumference prior to the test. The lower bearings shall consist of two wooden strips with vertical sides, each strip having its interior top corner rounded to a radius of

approximately ½ in. They shall be straight, and shall be securely fastened to a rigid block with their interior vertical sides one inch apart. The upper bearing shall be a wooden block, straight and true from end to end. The bearings shall be centered on the diametrically opposite markings previously made and the test load shall be applied through the upper bearing block in such a way as to produce a uniform distribution at the load throughout the length of the pipe and to leave the bearing free to move in a vertical plane passing midway between the lower bearings.

Application of Load (Machine Testing)

22. Any prime mover or hand power which will apply the load at a uniform rate of about 2000 lb. per minute, or in increments of not more than 100 lb., at the same rate, may be used in making the test. The testing machine shall be substantial and rigid throughout, so that the distribution of the load will not be affected appreciably by the deformation or yielding of any part. The load shall be applied continuously until the ultimate strength of the pipe is reached. The load at ultimate strength and also when the first crack appears shall be observed and recorded.

Pipe "out of line."

23. In testing pipe which is "out of line" the lines of the bearings chosen shall be from those which appear to give the most favorable conditions for fair test.

Test Result.

24. The test specimens shall show an ultimate strength not less than that given in Table I for the several classes and for the various sizes and methods of test stated and shall show no clearly visible crack extending the full length of pipe when tested to one-half the ultimate load designated in the Table.

TABLE I.—ULTIMATE STRENGTH TEST REQUIREMENTS OF REINFORCED-CONCRETE SEWER PIPE AT NOT LESS THAN 28 DAYS.

Internal Diameter of Pipe in inches.	Ultimate Load in pounds per lin. ft. of Pipe.			
	Class A Pipe.		Class B Pipe.	
	Three-edge Bearing.	Sand Bearing.	Three-edge Bearing.	Sand Bearing.
24	2400	3600	3600	5400
27	2650	3975	4000	6000
30	2900	4350	4350	6525
33	3150	4725	4700	7050
36	3400	5100	5000	7500
42	3800	5700	5550	8325
48	4200	6300	6000	9000
54	4500	6750	6400	9600
60	4800	7200	6750	10125
66	5000	7500	7000	10500
72	5200	7800	7200	10800

NOTE.—The load per foot of pipe shall be determined by dividing the total test load by the laying length of the pipe in feet.

VII. INSPECTION.

25. All pipe shall be subject to inspection at the factory, trench or other point of delivery by a competent inspector employed by the consumer or purchaser. The purpose of the inspection shall be to cull and reject pipe which, independent of the physical tests herein specified, fail to meet the requirements of these specifications.

26. Pipe shall be subject to rejection on account of any of the following:

(a) Variations in any dimension exceeding the permissible variations given in Section 13.

(b) Fractures or cracks passing through the shell, except that a single crack at either end of the pipe, not exceeding 2 inches in length, or a single fracture not exceeding 2 inches in depth, nor extending more than 10 per cent around the circumference, will not be considered cause for rejection, unless these defects exist in more than 5 per cent of the entire consignment.

(c) Defects which indicate imperfect mixing and molding.

(d) Exposure of the reinforcement when such exposure would indicate that the reinforcement is misplaced.

27. All rejected pipe shall be plainly marked by the inspector and shall be replaced by the manufacturer or seller with pipe which meets the requirements of these specifications, without additional cost to the consumer or purchaser.

DISCUSSION.

J. E. Longley.—I would like to present the views of the Lock Joint Pipe Company in regard to these specifications for reinforced-concrete sewer pipe. We are entirely opposed to their adoption. The specifications as proposed will fail to serve the purpose for which they are written. The purpose of a specification is to assist the manufacturers and consumers by defining the standard of the product which should be sold and used. They will fail because they do not define the standard which can be sold under the specifications. Any pipe, no matter what its wall thickness, its quantity of steel, its strength of concrete, can be sold under these specifications.

They are based on the idea that load tests alone are sufficient to protect the user and the industry. This is not so, because they do not serve to keep inferior pipe off of the job. As a matter of fact, they actually invite poor material on to the work. The result in that case, of course, is a loss to the user through delay and inconvenience if he waits for good pipe. He may be at such a stage of his work that he is obliged to use the pipe. In either case the failure of the pipe to meet the tests or its subsequent failure in the ground means loss to the consumer and damage to the reinforced-concrete sewer pipe industry as a whole.

These specifications encourage the manufacture and use of pipe that is likely to fail. We do not want to see failure. Every single failure to meet a load test or failure in service means serious damage to the industry. We want to see only such pipe included in these specifications as will stand up under the specified load tests and in service. This can be done only by specifying minimum amounts of steel, mix or strength of concrete and wall thickness which, when used with reasonable care, will make a pipe which is certain to pass the tests.

These specifications ignore definite standards which have already been established. Reinforced-concrete sewer pipe has been made and used in enormous quantities in this country for many years. The manufacturing concerns largely responsible for the development of reinforced-concrete pipe in this country based their standards on experimental tests and careful observation of pipe in service. Is it wise to ignore the fact that not a single length made under these standards has failed in service for a long period of years? It is this fact alone which has given the pipe a high place of merit in the minds of engineers and users. Study of material used in lines which have been in service for years will give a very close indication of the minimum amount of material which should be recommended by the Institute. If, in the future, we pipe manufacturers want to make pipe thinner or with less steel or cement, we should show conclusively that the product is still the standard acceptable to the American Concrete Institute and we can then apply for a reduction in the minimum

specified. Load tests alone are not sufficient for determining minimum wall thicknesses; they are not a complete criterion of suitability for the conditions to which the pipe will be subjected in the trench.

We, as manufacturers, are interested in seeing reinforced-concrete sewer pipe become a standard material of construction. The American Concrete Institute certainly wants to see the same thing, but it never will under specifications which allow manufacturers to make anything they wish. This specification would represent an American Concrete Institute license to manufacturers to make reinforced-concrete pipe with just as little material as they would dare.

Various members of the committee anticipate difficulty in arriving at minimum quantities of steel and minimum wall thicknesses, due to the fact that there are different methods of making pipe. If this is so and the Institutes recognizes different methods of manufacturing, then there should be separate specifications for the pipe made by each method. Pipe made by the poured process under field conditions forms by far the largest pipe output of the reinforced-concrete sewer pipe business. The specifications should therefore be governed by this class of pipe. If there are other processes to which the specifications would be unfair, then separate specifications or provisions should be made to take care of them.

The future of the reinforced-concrete sewer pipe industry depends on the degree of satisfaction which the product of the industry gives. In an organization such as the American Concrete Institute, made up of consumers, engineers and manufacturers who are all interested in the welfare of the product, in a case of this kind where specifications are being drawn up, it is the duty of the engineers to protect the consumers from the use of poor material, and it is the duty of the manufacturers to protect the consumers by preventing, as far as possible, the production of poor material. These specifications do not protect the consumer.

On the basis of these facts, we absolutely oppose the adoption of the specifications as they are written.

C. F. BUENTE.—I would like to reply briefly to the argument just placed before the convention. The question at issue is whether we shall include in our specifications a design of pipe. At a well attended meeting of the committee held at Chicago early last year, two specifications which had been previously prepared, were submitted for consideration of the committee. Both specifications contained complete designs for pipe and after full discussion by the committee, both specifications were rejected. The committee then authorized appointment of a sub-committee to prepare specifications to be submitted to the committee, giving the sub-committee, at the same time, instructions to prepare these specifications based on performance or test only. This sub-committee held a number of meetings at Chicago, submitting its work to the general committee for discussion,

and after the specifications were finally written, the work was placed before the general committee for letter ballot, resulting in ten favorable and only one unfavorable vote. Up to this time no criticism of the work of the sub-committee had been made, but at a meeting of the general committee held at Chicago last month some criticism of these specifications was submitted, based on the fact that they did not contain a design of pipe. With this criticism, however, no constructive work was offered as a substitute.

AMERICAN CONCRETE INSTITUTE STANDARD.

Standard Specifications for Portland Cement Concrete Pavements.*

One-Course Portland Cement Concrete Pavement for Highways.

I. GENERAL.

1. It is the intent of these specifications to cover the requirements for materials and construction of portland cement concrete pavement, wherein the concrete is of uniform proportions from top to bottom of the slab.

II. MATERIALS.

(A) *Cement.*

2. Cement shall be a standard portland cement which at the time it is incorporated in the pavement mixture, shall conform to the Standard Specifications and Tests for Portland Cement (Serial Designation: C9-21) of the American Society for Testing Materials, and subsequent revisions thereof. **Cement**

(B) *Aggregates.*

3. Prior to placing any orders for aggregates the contractor shall advise the engineer of the proposed source or sources of supply of aggregates. The engineer may require the contractor to submit 50-lb. samples of all aggregates proposed for use. If the engineer finds such samples fulfill the requirements of these specifications for aggregates, similar material shall be considered as acceptable. Acceptance of samples shall not be construed as a guarantee of acceptance of all materials from the same source, and it shall be understood that any aggregates which do not meet with the requirements of these specifications will be rejected. Upon receiv- **Aggregates.**

---

* Submitted by Committee S-6, on Concrete Roads and Pavements, for annual meeting, February, 1924. Tentative Standards were printed in *Proceedings*, A. C. I., Vol. 19, 1923, p. 387. Discussion of Specifications appears on p. 729 of this volume.

Adopted by meeting to be sent out to letter ballot as Standard Specification, Feb. 26, 1924.

Adopted as Standard Specification by letter ballot of the Institute, Apr. 29, 1924.

ing notification of the proposed source or sources of aggregate supply, the engineer may elect to investigate and test the aggregate supply at the source; in which case he shall notify the contractor as to acceptability, or non-acceptability of the proposed aggregates. The engineer shall notify the contractor, after agreement upon a source or sources of aggregate supply, whether routine tests of aggregates during construction will be made at the source of supply or at the point of receipt. (See Bidding Sheet.)

Fine Aggregate.

4. (*a*) *Fine Aggregate.*—Fine aggregate shall consist of natural sand, stone screenings, slag sand, tailings, chatts, or other inert materials with similar characteristics, or a combination thereof, having clean, hard, strong, durable, uncoated grains. When incorporated in the pavement mixture, fine aggregate shall be free from frost, frozen lumps, injurious amounts of dust, mica, soft or flaky particles, shale, alkali, organic matter, loam, or other deleterious substances. Ninety-five per cent of the fine aggregate, when dry, shall pass a one-fourth (¼) in. screen; not more than 25 per cent shall pass a 50-mesh sieve, and not more than 5 per cent by weight shall pass a 100-mesh sieve. In no case shall fine aggregate be accepted containing more than 3 per cent, by dry weight, nor more than 5 per cent by dry volume, nor more than 7 per cent by wet volume, of clay, loam, or silt. If any sample of fine aggregate shows more than 7 per cent of clay, loam or silt, in one hour's settlement after shaking in an excess of water, the material represented by the sample will be rejected.. Fine aggregate shall be of such a quality that mortar composed of portland cement and the fine aggregate when made into 2 x 4 in. cylinders, in the same proportions as will be used in the concrete mixture for the pavement, shall show compressive strength at 7 and 28 days equal to or greater than the compressive strength of cylinders composed of mortar of the same proportions of portland cement and standard Ottawa sand. For proportioning test cylinders, portland cement and fine aggregate and standard Ottawa sand shall be measured by weight, and the same portland cement shall be used with the Ottawa sand as with the fine aggregate to be tested.

Coarse Aggregate

5. (*b*) *Coarse Aggregate.*—Coarse aggregate shall consist of one of the following materials, or a combination thereof: crushed rock, pebbles (gravel), air cooled blast furnace slag, chatts or tailings. The particles of coarse aggregate shall be of clean, hard, tough, durable material, free from vegetable or other deleterious substances, and shall contain no soft, flat, or elongated pieces. Coarse aggregate shall show not more than 6 per cent loss in the wear test.

(Note: In many cases, it will be necessary for the engineer to specify the sizes, grading, and quality of coarse aggregate in accordance with local conditions. In every case, the engineer should provide specifications which will require the use of the best coarse aggregate which is economically available. As a guide to the engineer, Abrams' "Tables of Proportions and Quantities for Concrete for Road Construction" are printed herewith. The following specifications covering size and grading of coarse aggregate will

be found applicable in most sections of the country, and are intended for use with the 1: 2: 3½, 1: 2: 3, or 1: 1½: 3, mixture.)

6. The size of the coarse aggregate shall be such as to pass a 3-in. round opening. Coarse aggregate shall be uniformly graded within the limits shown in the following table, and any material which does not come within the limits specified shall be rejected.

Passing 3-in. round opening, 100 per cent.

Passing 2-in. round opening, not less than 82 per cent nor more than 95 per cent.

Passing ½-in. round opening, not less than 15 per cent nor more than 25 per cent.

Passing ¼-in. sieve, not more than 5 per cent.

7. *Crushed Rock* shall consist of particles of rock produced by quarrying and crushing ledge rock, field boulders, or pebbles, from which, after crushing, all dust and pieces below one-quarter (¼) in. size have been screened out. Crushed rock shall conform in quality to the specifications under "Coarse Aggregate." **Crushed Rock.**

8. *Pebbles* (*Gravel*) shall consist of loose material containing only particles retained upon a ¼-in. screen, resulting from the natural crushing and erosion of rocks. Pebbles must have wearing qualities at least equal to crushed stone. Pebbles shall conform in quality to the specifications under "Coarse Aggregate." **Pebbles.**

9. *Air Cooled Blast Furnace Slag.*—The broken slag shall consist of roughly cubical fragments of air cooled blast furnace slag, reasonably uniform in density and quality and reasonably free from metallic iron, containing no dirt or other objectionable matter. The slag shall weigh not less than seventy (70) pounds per cubic foot. **Slag.**

10. *Chatts,* or *Tailings* are terms locally applied to by-products, or waste products, of certain mining and industrial operations. When used as coarse aggregate for concrete pavements, such materials shall substantially conform to the specifications under "Coarse Aggregate." **Chatts.**

11. *Mixed Aggregate.*—Mixed aggregate shall consist of a combination of fine and coarse aggregates. That portion of mixed aggregate passing a one-quarter (¼) in. screen shall conform to the requirements for fine aggregate; and that portion of mixed aggregate retained on a one-quarter (¼) in. screen shall conform to the requirements for coarse aggregate. **Mixed Aggregate.**

(C) *Water.*

12. Water shall be clean, free from oil, acid, alkali, or vegetable matter. **Water.**

(D) *Reinforcement.*

Reinforcement. 13. Reinforcement shall consist of steel fabric, or of steel bars, or a combination of both and shall have an effective weight exclusive of dowel bars at joints and of circumferential bars of at least....pounds per 100 square feet.

14. (a) *Steel Fabric.*—Steel fabric shall be manufactured from cold drawn wire and shall comply with tentative standards of the American Society for Testing Materials, Serial Designation A 82-21T.

15. The spacing of primary members shall be not more than....inches, and of secondary members not more than....inches.

16. *Steel Bar Reinforcement.*—This style of reinforcement shall consist of steel bars of the size, shape and spacing shown on the plans, and shall be properly formed into mats. All intersections of longitudinal and transverse bars along the exterior edges of the mat and every other intersection of the longitudinal and transverse bars in the interior of the mat shall be securely wired or clipped together to resist displacement during handling and concreting operations. The materials shall have an effective weight of not less than....lb. per 100 sq. ft., exclusive of laps, ties, clamps, chairs, and such portions of the bars as are not in the plane of the mat for their full lengths.

17. Steel bars shall comply with the standard requirements for concrete reinforcement bars, structural and intermediate grades, of the American Society for Testing Materials, Serial Designation A 15-14.* All bar reinforcement when placed in the pavement shall be free from excess rust, scale, or other substance which prevents the bonding of the concrete to the reinforcement. When in storage on the work, bars shall be protected from corrosion by placing them on a dry platform under a weather-proof cover.

(E) *Joint Filler.*

Joint Filler. Joint filler shall consist of prepared strips of fibre matrix and bitumen, containing not more than 25 per cent of inert material, having thickness of.....in., and width equal to.....in. greater than the thickness of the pavement at any point. The bitumen used in manufacture of the joint filler may be either tar or asphalt of a grade that will not become soft enough to flow in hot weather, nor brittle in cold weather. The prepared strips shall be cut to conform to the cross-section of the pavement and in lengths equal to the width of the pavement, except that strips equal in length to half the width of this pavement may be used when laced or clipped together at the center in a workmanlike and effective manner.

(F) *Shoulders.*

Shoulders. (Any special materials for the construction of shoulders should be here described as desired by the engineer.)

---

* This clause, which restricts bar reinforcement to bars rolled from new billets, was referred back to the Committee for further investigation and report at the next annual meeting.

III. SUBGRADE.

18. *Subgrade* will be considered as that portion of the highway upon which the pavement is to be placed.

(A) *Fine Grading.*

19. Fine grading will include the finished excavation and embankment which may be necessary to bring the subgrade to the required elevation, alignment, and cross-section. All suitable materials removed from the excavation in fine grading shall be used as far as practicable in the formation of the embankment, as may be required. Such material not used in embankment may be deposited on the shoulders as directed by the engineer. When the amount of the embankment exceeds the amount of the material available from excavation, suitable material shall be obtained by the contractor from borrow pits located beyond the limits of the shoulders or embankment slopes. Such borrow pits shall be left in neat condition, such as will drain completely. Ditch sections and back slopes of cuts must conform to the plans, and be left with neat and uniform appearance. **Fine Grading.**

(B) *Preparation and Maintenance.*

20. The subgrade shall be constructed to have, as nearly as practicable, a uniform density throughout its entire width. Wherever the subgrade extends beyond the lateral limits of an old roadway, or wherever an old gravel, macadam, or other hard compacted crust comes within 6 in. of the elevation of the finished subgrade, such old roadway or crust shall be ploughed, loosened or sacrificed to a depth of at least 6 in. and the loosened material redistributed across the full width of the subgrade, adding suitable material, when necessary, so that when compacted to the required elevation, alignment, and cross-section, the subgrade will approach as nearly as possible, a condition of uniform density. Compression of the subgrade material shall be accomplished with a self-propelled roller weighing not less than 3 tons. Hand-tamping portions of the subgrade may be directed by the engineer when necessary. There shall not be left on the subgrade or shoulders, berms or ridges of earth or other material that will interfere with the immediate discharge of water from the subgrade to the side ditches, and the subgrade shall be maintained free from ruts so that it will, at all times, drain properly. **Preparation**

21. All depressions developing under traffic on the subgrade, or in connection with rolling, shall be filled with suitable material. Rolling shall be continued until the subgrade is uniformly compacted, properly shaped, and true to grade and alignment. It is not intended that the rolling shall be continued beyond this point, as the purpose of rolling is not to produce a subgrade that cannot be further compacted, but to produce a uniformly compacted subgrade. All hauling shall be distributed over the

width of the subgrade so far as practicable, so as to leave it in a uniformly compacted condition.

22. After being prepared in the above manner, the subgrade shall be so maintained until the concrete pavement has been placed thereon.

(C) *Subgrade—Checking and Acceptance.*

Checking. 23. Immediately prior to placing concrete pavement on the subgrade, it shall be checked by means of an approved scratch template, resting on the side forms, having the scratch points placed not less than 8 in. apart, and to the exact elevation and cross-section for the subgrade surface. The scratch template shall be drawn along the forms so that the plane of the points will be at a right angle to the grade line, and the long axis of the template at a right angle to the center line of the pavement. All high places indicated by the scratch points shall be removed to true grade, and any low places back filled with suitable material and rolled or hand tamped until smooth and firm. The subgrade shall be checked and completed in accordance with these requirements for a distance of not less than 100 ft. in advance of the concrete. If hauling over the subgrade after it has been finished and checked as above specified, results in ruts or other objectionable irregularities, the contractor shall re-roll or hand-tamp the subgrade and place it in smooth and satisfactory condition before the pavement is deposited upon it. If the condition of the subgrade is such that it cannot be placed in satisfactory condition to receive the pavement by the above methods, placing pavement may be stopped by the engineer unless the contractor can provide and haul over suitable trackways or use other satisfactory means for the protection and maintenance of the subgrade.

(D) *Special Treatment.*

24. (Special treatment may be specified for certain subgrades such as sand, gumbo, adobe, and other materials, which cannot be satisfactorily prepared for pavement by the methods specified in the foregoing paragraphs.)

IV. FORMS.

(A) *Materials.*

Materials. 25. Wooden forms shall be dressed to 3-in. thickness, and equal in depth to the thickness of the pavement at the sides. Forms shall rest upon stakes driven into the ground within 1 ft. of each end of each separate piece, and at intervals not greater than 5 ft. elsewhere. Forms shall be held in place by stakes driven into the ground along the outside edge at intervals of not more than 6 ft., two stakes being placed at each joint. The forms shall be firmly nailed to the side stakes, and firmly braced at any point where necessary to resist the pressure of the concrete or the

impact of the tamper. Forms shall be capped along the inside upper edge with 2-in. angle irons.

26. Metal forms shall be of shaped steel sections not less than 10 ft. in length, for tangents and for curves having radii of 150 ft. and over. For curves of less radii, sections 5 ft. long may be used. Forms must have a depth equal to the side thickness of the pavement. Forms shall be made of steel plate of approved section. At least three bracing pins or stakes shall be used to each 10 ft. of form, and the bracing and support must be ample to resist the pressure of the concrete and the impact of the tamper without springing.

(B) *Setting.*

27. Forms shall set to exact grade and alignment at least 500 ft. in advance of the ploint of depositing concrete. Before setting the sections must be thoroughly cleaned. After setting they shall be thoroughly oiled before concrete is placed against them. Forms in place will be subject to check and correction of line or grade at any time. **Setting Forms.**

V. PAVEMENT SECTION.

28. *Width, Thickness and Crown* of concrete pavement shall be as shown on the plans for the improvement. **Pavement Section.**

VI. JOINTS.

29. The joints to be formed shall be transverse or longitudinal. They shall be tested during and after finishing with a 10 ft. straight edge and any irregularities in the surface shall be immediately corrected. Expansion joints shall be formed between the pavement under construction and all other rigid types of pavement or structures to which it may be adjacent. All joints shall be edged to a radius of ⅛ in. Joints shall be made as follows: **Joints.**

(A) *Transverse Expansion Joints.*

30. Transverse expansion joints shall be....in. wide, spaced....ft. apart. A bulkhead cut to the exact cross-section of the pavement, shall be securely staked in place at right angles to the center line and surface of the pavement. The premolded joint filler shall be placed against the bulkhead and held in position by pins on which there is an outstanding lug. Concrete shall be deposited on both sides of the bulkhead before it is removed. After the concrete has been struck off the bulkhead shall be removed by lifting it slowly from one end and replacing it with concrete as it is lifted, so that the joint-filler will be left in the correct position.

31. When expansion joints are made at the end of the day's work they shall be formed by finishing the concrete to the bulkhead, placed as before

specified. When work is resumed the joint-filler shall be placed against the hardened concrete and held in position by pins until fresh concrete is placed against it.

32. In pavements with integral curb the joint shall be continuous in a straight line through pavement and curb.

33. Joints shall be opened on the edges for their entire depth, upon removal of the forms.

34. Before the pavement is opened to traffic the joint-filler shall be trimmed off to a uniform height of ¼ in. above the surface of the pavement.

(B) *Longitudinal Expansion Joints.*

Longitudinal Joints. 35. Longitudinal expansion joints shall be formed by placing the filler against the form, bulkhead, curb, or adjacent structure and placing the concrete against it. The filler shall extend the full depth of the pavement, and be flush with the pavement surface.

(C) *Transverse Construction Joints.*

Transverse Joints. 36. Transverse construction joints shall be formed whenever it is necessary to stop concreting for 30 min. or longer, except at expansion joints, by staking in place a bulkhead, as specified for transverse expansion joints, and finishing the concrete to the bulkhead. An edging tool shall be used along the bulkhead to make the construction joint a regular and well-defined line. When the plans require steel dowels across transverse joints in this bulkhead there shall be holes spaced 3 ft., center to center, 3 in. below the surface of the finished pavement, through which ¾ in. plain round steel rods 4 ft. long shall be inserted with 2 ft. projecting. At least one-half length of each bar shall be encased in heavy paper or coated with paint or oil in such a manner as to prevent a bond between the steel and the concrete.

37. When work is resumed the plank shall be removed, care being taken not to disturb the rods or the concrete. The fresh concrete shall be placed directly against the face of the concrete previously laid and carefully worked around the rods.

38. If concreting must be stopped within 10 ft. of a previously made transverse joint the concrete shall be removed to this joint.

(D) *Longitudinal Construction Joints.*

Construction Joints. 39. Longitudinal construction joints shall be formed where required and must be straight and vertical. When so indicated on the plans, steel dowels shall be used as provided in the preceding section.

VII. WATER SUPPLY.

(A) *Equipment.*

40. Where necessary for the supply of water for all operations described in these specifications, duplicate pumps, connected to an adequate pipe line along the improvement, shall be provided by the contractor. The pipe line must be fitted with drains at the low points, and air relief valves at the high points, and with convenient outlets for all paving operations. Where the concrete mixer operates on the subgrade, the pipe line shall have a minimum diameter of 2 in. For supplying a mixer using more than 4 sacks of cement per batch, 60 per cent of the pipe line shall have a minimum diameter of 3 in., and the remaining 40 per cent shall have a minimum diameter of 2 in. The large diameter pipe shall lead from the pump. Water Supply.

(B) *Priority to Water Supply.*

41. The concrete pavement in place, for 10 days after laying, and the subgrade preparation, shall have prior rights to the water supply. If it should develop there is not sufficient water for all purposes, the concrete mixer shall be shut down until the water needs of the curing and subgrading operations have been cared for.

VIII. PROPORTIONING AND MIXING CONCRETE.

(A) *Proportioning.*

42. (a) *Measuring Materials.*—The method of measuring the materials for the concrete including water, shall be such as to insure the required proportions of each of the materials as directed by these specifications. One sack of portland cement (94 lb. net) shall be considered 1 cu. ft. Measuring Materials.

43. (b) *Proportions.*—The concrete shall be proportioned 1 sack of portland cement, not more than......cu. ft. of fine aggregate, and not more than......cu. ft. of coarse aggregate. A cu. yd. of concrete in place, measured between neat lines, must contain......barrels of portland cement. The engineer shall compare the calculated amount of cement required by these specifications and tables with the amounts actually used in each section of concrete......ft. long, or between successive transverse joints. If the amount of cement actually used in the pavement varies from the specified amount by more than 3 per cent for any section, the engineer may require the proportions of the concrete to be adjusted so as to use the specified amount of cement. If it is found that the amount of cement used in any section is 92½ per cent or less, of the specified quantity, the contractor shall be required to remove such section or sections, and replace them with concrete made in accordance with these specifications. Such removal and replacement shall be done at the expense of the contractor. Proportions

(B) *Mixing.*

Mixing.

44. (a) *Operation of Mixer.*—The concrete shall be mixed in a batch mixer, with the "boom and bucket" type of delivery. The capacity of the drum shall be such that only whole bags of cement are used in each batch. Mixing shall continue for at least 1 min. after all materials, including water, are placed in the drum, and before any part of the batch is discharged. The drum shall be revolved not less than 14 nor more than 18 revolutions per minute. The drum shall be completely emptied before receiving materials for the succeeding batch. The volume of the mixed material in each batch shall not exceed the mixer manufacturer's rated capacity of the drum.

45. The mixer shall be provided with a water measuring tank into which mixing water shall be discharged, having a visible gauge so that the amount of water for each batch may be separately and accurately measured. The mixer shall be provided with an approved batch timing device which will automatically lock the batch discharging device during the full mixing time and release it at the end of the mixing period. The timer device shall have a bell which will automatically ring at the end of the mixing period. This device shall be subject to inspection and adjustment by the engineer at any time.

Retempering

46. (b) *Retempering.*—Mortar or concrete which has partially set shall not be retempered by being mixed with additional materials or water.

Central Mixing.

47. (c) *Central Mixing Plants.*—The use of central mixing plants and the transportation of mixed concrete is permitted under these specifications, provided there is no segregation of the mixed concrete when it is delivered at the point where it is to be deposited in the pavement. The period between mixing and placing in the pavement shall not exceed 40 min., and this period may be reduced at the direction of the engineer. The concrete must be of workable consistency when placed on the subgrade.

Consistency.

48. (d) *Consistency.*—The concrete mixture shall contain no more water than is necessary to produce a workable mass which can be brought to a satisfactory finish in the pavement. The amount of water used shall not exceed 6¼ gal. per sack of cement, when the aggregates are dry.

IX. PLACING CONCRETE AND REINFORCEMENT.

(A) *Inspection of Subgrade.*

48. (a) *Rechecking Subgrade.*—Immediately before placing concrete, or any type of reinforcement, the subgrade shall be rechecked by means of a scratch template as provided in paragraph 23 of these specifications, and any inequalities corrected as therein provided.

50. (b) *Condition of Subgrade.*—Concrete shall be placed only on a moist subgrade, but there shall be no pools of standing water. If the sub-

grade is dry, it shall be sprinkled with as much water as it will absorb readily. The engineer may direct that the subgrade may be sprinkled or thoroughly wet down from 12 to 36 hours in advance of placing concrete, where such procedure may be deemed necessary.

(B) *Placing Reinforcement.*

51. Steel fabric reinforcement of the size and weight shown on the plans, shall be placed 2 in. below and parallel to the finished surface of the pavement unless otherwise indicated. Fabric shall extend to within 2 in. of sides and ends of slabs. All laps of fabric sections shall be not less than three-fourths of the spacing of members in the direction lapped. Steel bar reinforcement shall be placed 3 in. below the finished surface of the pavement unless otherwise indicated on the plans. Transverse bars shall extend to within 2 in. of the margins of the pavement. Bar reinforcement shall be placed and securely supported in correct position before any concrete is laid. All intersections of longitudinal and transverse bars shall be securely wired or clipped together to resist displacement during concreting operations. **Placing Reinforcement.**

(C) *Placing Concrete.*

52. The mixed concrete shall be deposited rapidly on the subgrade to the required depth and for the entire width of the pavement section, in successive batches and in a continuous operation without the use of intermediate forms or bulkheads between joints. While being placed, the concrete shall be vigorously sliced and spaded with suitable tools to prevent formation of voids or honeycomb pockets. The concrete shall be especially well spaded and tamped against the forms. When the concrete is placed in two horizontal layers to permit use of steel reinforcement, the first layer shall be roughly struck off with a template or screed, riding on the side forms, at the correct elevation to permit placing the reinforcement in specified position. The concrete above the reinforcement shall be placed within 15 min. after the first layer has been placed. Any dust, dirt or foreign matter which collects on the first layer shall be carefully removed before the upper layer is placed. **Placing Concrete.**

53. In case of a breakdown of the mixer, involving stopping operations for more than 40 min., a transverse joint shall be formed at the point directed by the engineer, to close the section. Any concrete in excess of that needed to complete a section, when work is stopped for more than 40 min. shall not be used in the pavement.

(D) *Finishing.*

54. (a) *General.*—Experienced and skillful workmen must be employed at all times for preparing the surface of the pavement. The concrete shall be brought to the specified contour by means of a heavy screed **Finishing.**

or template, fitted with handles, weighing not less than 15 lb. per lin. ft. This screed or template may be of steel, or of wood shod with steel. It shall be shaped to the cross-section of the pavement, and have sufficient strength to retain its shape under all working conditions. The template or screed shall rest on the side forms and shall be drawn ahead with a sawing motion. At transverse joints, the template shall be drawn not closer than 3 ft. toward the joint, and shall then be lifted and set down at the joint and drawn backwards away therefrom. Surplus concrete shall then be taken up with shovels and thrown ahead of the joint.

Belting.

55. (b) *Belting.*—The concrete shall be finished by using a belt of wood, canvas, or rubber, not less than 6 nor more than 12 in. wide, and at least 2 in. longer than the width of the pavement. The belt shall be applied with a combined crosswise and longitudinal motion. For the first application vigorous strokes at least 12 in. long shall be used, and the longitudinal movement along the pavement shall be very slight. The second application of the belt shall be immediately after the water sheen disappears, and the stroke of the belt shall be not more than 4 in. and the longitudinal movement shall be greater than for the first belting.

Machine Finish.

56. (c) *Machine Finishing.*—When a finishing machine is used it shall be so designed and operated as to strike off and consolidate the concrete, eliminating ridges and producing a true and even surface. The operation of the machine shall be so controlled as to keep the coarse aggregate near the finished surface of the pavement. Repeated operation of the machine over a given area is to be avoided.

57. A hand tamping template and belt must be kept for use in case the tamping machine breaks down.

Floating.

58. (d) *Longitudinal Floating.*—Immediately after the screeding specified under IX (D) 54 (a) has been completed, the surface should be inspected for high or low spots and any needed corrections made by adding or removing concrete. Rough spots should be gone over with a long handled float and worked to proper contour and grade. The entire surface shall then be floated longitudinally, with a float board not less than 16 ft. long and 8 in. wide. This float board shall have convenient plow-handles at each end. It shall be operated by two men, one at each end, each man standing on a bridge spanning the pavement. The lower surface of the float board shall be placed upon the surface of the concrete with the long dimension parallel to the center line of the pavement. The float shall then be drawn back and forth in slow strokes about 2 ft. long, and advancing slowly from one side of the pavement to the other. The purpose of this operation is to produce a uniform even surface on the concrete, free from transverse waves. The two bridges on which the workmen stand should be placed about 18 ft. apart when the length of the float is 16 ft. When the entire width of the pavement has been floated in this manner from one position of the bridges, they shall be moved ahead about 12 ft. so that the

next section to be floated shall overlap the one previously so floated from 3 to 4 ft. After this floating has been completed, and all transverse waves eliminated, the surface shall be finished by the belting process specified in paragraph 55.

59. (e) *Finishing at Joints and Tooling.*—The contractor shall provide a suitable split float or split roller, having a slot to fit over expansion joints. This device shall be so arranged as to float the surface for a width of at least 3 ft. on each side of the joint simultaneously. This device shall be used in such manner as to produce a true surface across the joint. Edges of the pavement, at joints and side shall be tooled for a width of 2 in., the corners rounded to a radius of ¼ in. **Finishing at Joints.**

60. (f) *Trueness of Surface.*—The finished surface of the pavement must conform to the grade, alignment and contour shown on the plans. Just prior to the final finishing operation, the surface shall be tested with a light straight edge, 10 ft. in length, laid parallel to the center line of the pavement. Any deviation shall be immediately corrected. **Trueness of Surface**

61. The contractor shall be held responsible for the trueness of surface of the pavement, and shall be required to make good any deviation from the alignment, grade, and contour shown on the plans, in excess of the tolerance stipulated in this section.

X. CURING AND PROTECTION.

(A) *Burlap Cover.*

62. The contractor shall provide a sufficient amount of burlap or canvas for every mixer on the job, to cover all of the pavement laid in any one day's maximum run. Burlap or canvas cover shall be made up in sheets 12 ft. wide, and 4 ft. longer than the width of the pavement. Burlap or canvas cover shall be placed on the concrete immediately after the final belting, and shall then be sprayed with water in such a manner that the surface of the pavement will not be damaged. Burlap or canvas cover shall be kept continuously moist by spraying until the concrete has taken final set. **Burlap Cover**

(B) *Wet Earth Cover.*

63. As soon as it can be done without damaging the concrete, the surface of the pavement shall be covered with not less than 2 in. of earth, or 6 in. of hay or straw. This cover shall be kept continuously wet by spraying for 10 days after the concrete is laid. **Wet Earth Cover.**

(C) *Sprinkling or Ponding.*

64. The sprinkling system of curing may be used if approved by the engineer. The sprinkling equipment shall be placed carefully, and without **Sprinkling or Ponding.**

injuring the concrete surface. The sprinkling system shall be so arranged, and supplied with sufficient water at ample pressure, to keep every portion of the pavement surface continuously wet (both night and day) for 10 days after laying the concrete. Dikes shall be constructed along both edges of the pavement, with cross-dikes where necessary, and the water flowing off the surface of the pavement shall be collected and led to the ditches of culverts as directed by the engineer. The contractor shall be held responsible for any damage to the roadway, shoulders, or adjacent property, by reason of escaping water.

65. The ponding system of curing may be used at the option of the contractor. Dikes shall be built along both edges of the pavement, with cross-dikes at sufficiently frequent intervals, and the pavement flooded with sufficient water within the dikes to keep all portions of the pavement surface continuously covered with water for 10 days after the concrete is laid.

(D) *Cleaning.*

Cleaning. 66. After 14 days, the earth or other cover may be removed. After 30 days, the contractor may use a mormon or a fresno scraper to remove the cover, except that scrapers shall not be used within 1 ft. of expansion joints. Cover within 1 ft. of expansion joints must be removed by hand. Road machines, or blade graders of the 2- or 4-wheel type shall not be used for removing the cover.

67. After the cover has been removed, or ponds emptied and dikes removed, the entire surface of the pavement shall be swept clean and free from dirt and debris. Horse or motor drawn sweepers shall not be operated on the pavement till 30 days have elapsed after the concrete is placed.

(E) *Cold Weather Protection.*

Cold Weather Protection. 68. Concrete shall not be mixed nor deposited when the temperature is below freezing, except under such conditions as the engineer may direct in writing. If, at any time during the progress of the work, the temperature is, or in the opinion of the engineer, will, within 24 hours, drop to 38 deg. F., the water and aggregates shall be heated, and precautions taken to protect the concrete from freezing until it is at least 10 days old. In no case shall concrete be deposited upon a frozen subgrade, nor shall frozen materials be used in the concrete.

XI. PROHIBITION OF TRAFFIC.

(A) *Barricades.*

Barricades. 69. The contractor shall provide and maintain substantial barricades across the pavement, with suitable warning signs by day and by night, to prevent traffic of any kind upon the pavement before it is 21 days old, or

before the cover has been removed. The contractor shall provide and maintain watchmen at each mixer, whenever the paving crew is not at work, who shall prevent destruction or removal of barricades, and keep traffic off the pavement.

70. No section of pavement shall be opened to traffic until written instructions have been given by the engineer.

(B) *Crossings.*

71. At public highway and private crossings, the contractor shall provide suitable structures to carry the traffic across the pavement without injury to the concrete. All such structures shall be subject to the approval of the engineer, and he may direct their improvement, or repair, as conditions may require. **Crossings.**

XII. CONDITION BEFORE ACCEPTANCE.

72. Before the road will be considered completed in accordance with these specifications, and acceptable to the engineer, the pavement, shoulders, ditches, back slopes, and structures, shall be placed in a neat and orderly condition, conforming to the plans and specifications in all respects. Equipment, surplus materials, and construction debris of every description shall be removed from the right of way. **Condition Before Acceptance.**

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Standard Specifications for Portland Cement Concrete Pavements.*

Two-Course Portland Cement Concrete Pavement for Highways.

I. GENERAL.

1. It is the intent of these specifications to cover the requirements for materials and construction of Portland Cement Concrete Pavement composed of two layers of concrete made with unlike coarse aggregates, but of the same proportions.

II. MATERIALS.

Materials.

2. The requirements for

(A) *Cement,*

(B) *Aggregates,*

Fine Aggregate.

(a) *Fine Aggregate,*

shall be as specified in Section II, paragraphs (A), (B) and (a) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

Coarse Aggregate.

3. (b) *Coarse Aggregate for Bottom Course.*—Structurally sound material considered too soft for a pavement surface may be used as the coarse aggregate in the bottom course. It shall consist of crushed rock, pebbles (gravel), air cooled blast furnace slag, chatts or tailings. The particles of coarse aggregate shall be of clean, durable material, free from vegetable or other deleterious substances and shall contain no flat or elongated pieces.

(Note.—In many cases it will be necessary for the engineer to specify the sizes, grading, and quality of coarse aggregate in accordance with local conditions. In every case, the engineer should provide specifications which will require the use of the best coarse aggregate which is economically available. The following specifications covering size and grading of coarse aggregate will be found applicable in most sections of the country and are intended for use with proportions from 1 : 2 : 4 to 1 : 1½ : 3).

---

* Submitted by Committee S-6, on Concrete Roads and Pavements, for annual meeting, February, 1924. Accepted for Tentative Standard by meeting, February 26, 1924.

4. The size of the coarse aggregate shall be such as to pass a 3 in. round opening. Coarse aggregate shall be uniformly graded within the limits shown in the following table, and any material which does not come within the limits specified shall be rejected.

Passing 3-in. round opening, 100 per cent.

Passing 2-in. round opening, not less than 82 per cent nor more than 95 per cent.

Passing ½-in. round opening, not less than 15 per cent nor more than 25 per cent.

Passing ¼-in. sieve, not more than 5 per cent.

5. (c) *Coarse Aggregate for Top Course* shall consist of crushed rock, pebbles (gravel), air cooled blast furnace slag, chatts or tailings. The particles of coarse aggregate shall be of clean, hard, tough, durable material, free from vegetable or other deleterious substances and shall contain no soft or elongated pieces. The crushed rock shall wear not more than 6 per cent when subjected to the standard Deval abrasion test. When subjected to the abrasion test described on p. 30, U. S. Department of Agriculture, Bulletin 555, pebbles shall show a loss of not more than 12 per cent. **Top Course Aggregate.**

6. The size of the particles shall be such that at least 95 per cent shall pass a 1-in. round opening and not more than 5 per cent shall pass a ¼-in. sieve, with all the intermediate sizes retained.

7. The requirements for Crushed Rock, Pebbles (Gravel), Air Cooled Blast, Furnace Slag and Chatts or Tailings shall be as specified in Section II (*B*) (*b*) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

8. The requirements for

(d) *Mixed Aggregate,*

(C) *Water,*
(D) *Reinforcement,*
(E) *Joint Filler,*
(F) *Shoulders,*

shall be as specified in Section II, (B), (C), (D), (E), (F) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

III. SUBGRADE.

9. The requirements for subgrade, including **Subgrade.**

(A) *Fine Grading,*
(B) *Preparation and Maintenance of Subgrade,*
(C) *Checking and Acceptance of Subgrade,*
(D) *Special Treatment,*

shall be as specified in Section III, (A), (B), (C), (D) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

Forms.

IV. FORMS.

10. The requirements for

(A) *Materials,*
(B) *Setting,*

shall be as specified in Section IV, (A) and (B) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

Pavement Section.

V. PAVEMENT SECTION.

11. *Width and Thickness* of concrete pavement and the *depth* of the top and bottom courses shall be as shown on the plans for the improvement.

Joints.

VI. JOINTS.

12. The requirements for joints, including

(A) *Transverse Expansion Joints,*
(B) *Longitudinal Expansion Joints,*
(C) *Transverse Construction Joints,*
(D) *Longitudinal Construction Joints,*

shall be as specified in Section VI, (A), (B), (C), (D) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

Water Supply.

VII. WATER SUPPLY.

13. The requirements for

(A) *Equipment,*
(B) *Priority to Water Supply,*

shall be as specified in Section VII, (A) and (B) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

Proportioning

VIII. PROPORTIONING AND MIXING CONCRETE.

(A) *Proportioning.*

14. (a) *Measuring Materials.*—The method of measuring the materials for the concrete, including water, shall be such as to insure the required proportions of each of the materials as directed by these specifications. One sack of portland cement (94 lb. net) shall be considered 1 cu. ft.

15. (b) *Proportions.*—The concrete in both the top and bottom course shall be proportioned 1 sack of portland cement, not more than.....

cu. ft. of fine aggregate, and not more than......cu. ft. of coarse aggregate. A cu. yd. of concrete in place, measured between neat lines, must contain......barrels of portland cement. The engineer shall compare the calculated amount of cement required by these specifications and tables with the amounts actually used in each section of concrete....ft. long, or between successive transverse joints. If the amount of cement actually used in the pavement varies from the specified amount by more than 3 per cent for any section, the engineer may require the proportions of the concrete to be adjusted so as to use the specified amount of cement. If it is found that the amount of cement used in any section is 92½ per cent or less, of the specified quantity, the contractor shall be required to remove such section or sections, and replace them with concrete made in accordance with these specifications. Such removal and replacement shall be done at the expense of the contractor.

16. The contractor may, at his option, construct the top course of mortar composed of cement and fine aggregate mixed in the proportion of 1 sack of cement to......cu. ft. of fine aggregate.

17. The requirements for

(B) *Mixing.* **Mixing.**

(a) *Operation of Mixer,*
(b) *Retempering,*
(c) *Central Mixing Plants,*
(d) *Consistency,*

shall be as specified in Section VIII, paragraphs (B) (a), (b), (c), (d) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

IX. PLACING CONCRETE AND REINFORCEMENT. **Placing.**

18. The requirements for

(A) *Inspection of Subgrade.*

shall be as specified in Section IX, (A) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

(B) *Placing Reinforcement.* **Reinforcement.**

19. Steel fabric reinforcement of the size and weight shown on the plans shall be placed between the bottom and top courses, unless otherwise indicated. Fabric shall extend to within 2 in. of sides and ends of slabs. All laps of fabric sections shall be not less than the spacing of members in the direction lapped. Steel bar reinforcement shall be placed between the top and bottom courses unless otherwise indicated on the plans. Transverse bars shall extend to within 3 in. of the margins of the pavement. Bar reinforcement shall be placed and securely supported in correct posi-

tion before any concrete is laid. All intersections of longitudinal and transverse bars shall be securely wired or clipped together to resist displacement during concreting operations.

Placing Concrete.

(C) *Placing Concrete.*

20. The mixed concrete shall be deposited rapidly on the subgrade to the required depth and for the entire width between longitudinal joints, without the use of intermediate forms or bulkheads between joints. While being placed the concrete shall be vigorously sliced and spaded with suitable tools, to eliminate voids or honeycomb pockets. The concrete shall be especially well spaded and tamped adjacent to forms, bulkheads and curbs. The bottom course shall be struck off at the correct elevation with a template, or screed riding on the side forms. The top course shall be placed within 15 min. after the bottom course was placed. Any dust, dirt or foreign matter which collects on the surface of the bottom course shall be carefully removed before the top course is placed.

21. Whenever, because of a break-down or for any other reason operations will be stopped for more than 40 min., a transverse joint shall be formed at the point directed by the engineer, to close the section. Both the top and bottom courses shall be completed to this joint. Any concrete in excess of that needed to complete a section, when work is stopped for more than 40 min., shall not be used in the pavement.

22. The requirements for

Finishing.

(D) *Finishing.*

(a) *General,*
(b) *Belting,*
(c) *Machine Finishing,*
(d) *Longitudinal Floating,*
(e) *Finishing at Joints and Tooling,*
(f) *Trueness of Surface,*

shall be as specified in Section IX, (D) (a), (b), (c), (d), (e), (f) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

Curing.

X. CURING AND PROTECTION.

23. The requirements for

(A) *Burlap Cover,*
(B) *Wet Earth,*
(C) *Sprinkling or Ponding,*
(D) *Cleaning,*
(E) *Cold Weather Protection,*

shall be as specified in Section X, (A), (B), (C), (D), (E) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

XI. PROHIBITION OF TRAFFIC.

24. The requirements for

(A) *Barricades,*
(B) *Crossings,*

shall be as specified in Section XI, (A) and (B) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

XII. CONDITION BEFORE ACCEPTANCE.

25. The Condition before Acceptance shall be as specified in Section XII, of Specifications for One Course Portland Cement Concrete Pavement for Highways.

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Standard Specifications for Portland Cement Concrete Pavements.

One-Course Portland Cement Concrete Street Pavement.*

I. GENERAL.

1. It is the intent of these specifications to cover the requirements for the materials and construction of Portland Cement Concrete Street Pavement wherein the concrete is of uniform proportion from top to bottom of slab.

II. MATERIALS.

2. Materials shall meet the requirements given in the Specifications for One Course Portland Cement Concrete Pavement for Highways under Section II, Materials, in paragraphs

(A) *Cement,*
(B) *Aggregates,*
(C) *Water,*
(D) *Reinforcement,*
(E) *Joint Filler.*

III. SUBGRADE.

3. Subgrade will be considered as that portion of the highway upon which the pavement is to be placed.

(A) *Preparation and Maintenance.*

4. Preparation and Maintenance shall meet the requirements given in III Subgrade, paragraph (B) of Specifications for One Course Portland Cement Concrete Pavements for Highways.

---

* Submitted by Committee S-6, on Concrete Roads and Pavements, for annual meeting, February, 1924. Accepted for Tentative Standard by meeting, February 26, 1924.

(B) *Checking and Acceptance.*

5. Just before the concrete pavement is placed the subgrade shall be checked by means of an approved scratch template, when the crown is secured by screeding, or with stakes or "T's" when the crown is obtained by luting. All high places shall be removed to true grade and any low places filled with suitable material and rolled or hand-tamped until smooth and firm. The subgrade shall be checked and completed in accordance with these requirements for a distance of not less than 100 ft. in advance of the concrete. If hauling over the subgrade after it has been finished and checked as above specified results in ruts or other objectionable irregularities, the contractor shall re-roll or hand-tamp the subgrade and place it in smooth and satisfactory condition before the pavement is deposited upon it. If the condition of the subgrade is such that it cannot be placed in satisfactory condition to receive the pavement, placing pavement may be stopped by the engineer unless the contractor can provide and haul over suitable trackways or use other means for the protection and maintenance of the subgrade.

(C) *Aggregates on the Subgrade.*

6. Neither fine nor coarse aggregates shall be deposited upon the subgrade before the subgrade has been shaped and brought to the true finish.

(D) *Special Treatment.*

7. (Special treatment may be specified for certain subgrades such as sand, gumbo, adobe, and other materials, which cannot be satisfactorily prepared by the methods specified in the foregoing paragraphs.)

IV. ADJUSTING STREET STRUCTURES.

8. *Manhole and Catchbasin Covers*, valve boxes and similar existing structures within the area to be paved shall be adjusted by the contractor to come flush with the pavement surface.

V. FORMS.

(A) *Materials.*

10. (a) *Wooden Forms* shall be straight, dressed on at least one side, not less than 2 in. in thickness and equal in width to the depth of the concrete which is to be placed against them. Wooden forms shall be held in place by stakes driven into the ground along the outside edge at intervals of not more than 6 ft., two stakes being placed at each joint. The forms shall be firmly nailed to the side stakes and braced to resist the pressure of the concrete or the impact of tamping.

11. (b) *Metal Forms* shall be of shaped steel sections. They shall be straight, have a depth equal to the depth of the concrete to be placed against them and sufficient strength to resist without springing the working strains to which they are subjected.

(B) *Setting.*

12. Forms shall be set to exact grade and alignment at least 200 ft. in advance of the point where concrete is being deposited. Forms shall be thoroughly cleaned and oiled before concrete is placed against them. Forms in place will be subject to check and correction of line or grade at any time.

VI. JOINTS.

(A) *General.*

13. The joints to be formed shall be transverse or longitudinal. They shall be tested during and after finishing with a 10-ft. straight edge and any irregularities in the surface shall be immediately corrected.

14. All joints shall extend through the entire thickness of the pavement; shall be perpendicular to the surface of the pavement, and be edged to a radius of ¼-in.

(B) *Transverse Expansion Joints.*

15. Transverse expansion joints shall be placed across the pavement perpendicular to the center line. They shall be ⅜ in. wide and spaced 35 ft. apart between intersections. A bulkhead, cut to the exact cross-section of the pavement, shall be securely staked in place at right angles to the center line and surface of the pavement. The premolded joint filler shall be placed against the bulkhead and held in position by pins on which there is an outstanding lug. Before the bulkhead is removed concrete shall be deposited on both sides of it. After the concrete has been brought to the proper crown the bulkhead shall be removed by lifting it slowly from one end and replacing it with concrete as it is lifted, so that the joint filler will be left in the correct position.

16. When expansion joints are made at the end of the day's work they shall be formed by finishing the concrete to the bulkhead, placed as before specified. When work is resumed the joint filler shall be placed against the hardened concrete and held in position by pins until fresh concrete is placed against it.

17. In pavements with integral curb the joints shall be continuous in a straight line through pavement and curb.

18. Expansion joints in intersections shall be located as shown on the plans or as directed by the engineer.

19. Before the pavement is opened to traffic the joint filler shall be trimmed off to a uniform height of ¼ in. above the surface of the pavement.

20. The requirements for longitudinal expansion joints shall be as specified in VI Joints, paragraph (B) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

(C) *Longitudinal Expansion Joints.*

21. (D) *Transverse Construction Joints.*

The requirements for Transverse Construction Joints shall be as specified in VI Joints, paragraph (C), of Specifications for Portland Cement Concrete Pavements for Highways.

(E) *Longitudinal Construction Joints.*

22. These joints, located as shown on the plans, shall divide all pavements 30 ft. or more in width.

When the whole width of the pavement is placed in one operation longitudinal joints shall be formed by embedding in the pavement a strip of galvanized or painted 18 gauge, corrugated sheet metal 1 in. less in depth than the depth of the pavement at the joint. The metal shall be accurately staked to line and grade by means of pins driven vertically through holes provided for that purpose at not more than 4 ft. intervals. The pins shall be of mild steel ¼ in. in diameter and at least 15 in. long, and shall be left in place. The metal strips may be of any length. Punched or cut holes shall be provided in the strips at 5 ft. intervals to receive ½ in. round tie bars. The metal shall be carefully placed in the proper location and be vertical when the concrete is deposited about it.

23. When the pavement is built in successive longitudinal strips longitudinal joints shall be formed by painting the edge of sections first built with a bituminous paint and depositing the concrete in adjacent section against the painted edge.

24. Deformed bars having a sectional area of not less than 0.196 sq. in. and 4 ft. long shall be placed at 5 ft. intervals across all longitudinal joints, extending 2 ft. into the concrete on either side of the joint.

(F) *Manhole and Catch-basin Covers.*

25. Manhole and catch-basin covers and all other fixed objects in the pavement shall be separated from the concrete by joint filler.

VII. WATER SUPPLY.

(A) *From City Mains.*

26. Water taken from the city mains shall be paid for by the contractor at the rate of...... per thousand gallons.

(B) *Priority to Water Supply.*

27. The concrete pavement in place, for ten days after laying, and subgrade preparation, shall have prior rights to the water supply. If it should develop that there is not sufficient water for all purposes the concrete mixer shall be shut down until the water needs of the curing and subgrading operations have been cared for.

VIII. PROPORTIONING AND MIXING CONCRETE.

28. The requirements for

(A) *Proportioning.*

(a) *Measuring Materials,*
(b) *Proportions.*

(B) *Mixing.*

(a) *Operation of Mixer,*
(b) *Retempering,*
(c) *Central Mixing Plants,*
(d) *Consistency,*

shall be as specified under VIII, Proportioning and Mixing Concrete, in the Specifications for One Course Portland Cement Concrete Pavements for Highways.

IX. PLACING CONCRETE AND REINFORCEMENT.

(A) *Inspection of Subgrade.*

29. (a) *Re-checking Subgrade.*—Immediately before placing concrete, or any type of reinforcement, the subgrade shall be re-checked by the means provided in Section III (B) of these specifications.

30. The requirements for

(b) *Condition of Subgrade,*

(B) *Placing Reinforcement,*

shall be as specified in Section IX, (b) (B) Specifications for One Course Portland Cement Concrete Pavements for Highways.

(C) *Placing Concrete.*

31. The mixed concrete shall be deposited rapidly on the subgrade to the required depth and for the entire width between longitudinal joints in successive batches and in a continuous operation without the use of intermediate forms or bulkheads between joints. While being placed, the con-

crete shall be vigorously sliced and spaded, with suitable tools, to eliminate voids or honeycomb pockets. The concrete shall be especially well spaded and tamped against forms, bulkheads, curbs and gutters. When the concrete is placed in two layers to permit the use of steel reinforcement, the first layer shall be roughly struck off with a template or lute at the correct elevation to permit placing the reinforcement in the specified position. The concrete above the reinforcement shall be placed within 15 min. after the first layer has been placed. Any dust, dirt or foreign matter which collects on the first layer shall be carefully removed before the upper layer is placed.

32. Whenever the placing of concrete is to be suspended for more than 40 min., a transverse joint shall be formed, at the point directed by the engineer, to close the section. Any concrete in excess of that needed to complete a section, when work is stopped for more than 40 min., shall not be used in the pavement.

X. FINISHING.

(A) *Leveling Surface.*

33. (a) Experienced and skillful workmen shall be employed at all times for preparing the surface of the pavement. Between intersections the concrete shall be brought to the specified contour by means of a screed or template, fitted with handles, and weighing not less than 15 lb. per lin. ft. This template may be of steel, or of wood shod with steel. It shall be shaped to the contour of the pavement and have sufficient strength to retain its shape under all working conditions.

34. The template shall rest on the side forms, curbs or gutters and shall be drawn forward with a sawing motion. At transverse joints the template shall be drawn not closer than 3 ft. toward the joint, and shall then be lifted and set down at the joint and drawn backward away therefrom. Surplus concrete shall then be taken up with shovels.

35. After the concrete has been struck off the template shall be used as a tamp. In this operation one end of the template shall rest on the side support, while the other is lifted and dropped, advancing at such a rate that the whole pavement is struck at least once. The opposite end shall then be lifted and dropped and advanced in the same manner. In no case shall either end be advanced more than 1 ft. ahead of the other.

36. Or the correct pavement contour may be secured by the use of a lute. In that case steel grade stakes provided with lugs shall be driven into the subgrade with the top of the lugs accurately set at the elevation for the finished pavement. These stakes shall be set at intervals of 10 ft. along the subgrade and in a straight line perpendicular to the center line of the pavement, one at the center line and one at each quarter point and

at as many additional points as the engineer may direct. The concrete shall then be spread to the elevation indicated by the stakes.

37. Cement mortar gathered from the surface of the concrete already placed shall not be used in filling boot tracks or stony areas, but such imperfections shall be dug out and refilled with concrete to the depth of the reinforcing and worked smooth. No workmen shall then be allowed to walk over the area so completed.

38. The requirements for

(B) *Belting,*
(C) *Finishing at Joints and Tooling,*
(D) *Longitudinal Floating,*
(E) *Trueness of Surface,*

shall be as specified in Section IX, paragraphs (D), (b), (d), (e), (f), of Specifications for One Course Portland Cement Concrete Pavements for Highways.

XI. CURING AND PROTECTION.

39. The requirements for

(A) *Burlap Cover,*
(B) *Wet Earth,*

shall be as specified in Section X (A) and (B) of Specifications for One Course Portland Cement Concrete Pavements for Highways.

(C) *Sprinkling.*

40. The sprinkling system of curing may be used if approved by the Board of Local Improvements. The sprinkling equipment shall be placed carefully, and without injuring the concrete surface. The sprinkling system shall be arranged, and supplied with sufficient water at ample pressure, to keep every portion of the pavement surface continuously wet (both night and day) for 10 days after the concrete is laid. Water flowing off the surface of the pavement shall be led to the regular drainage system. The contractor shall be responsible for any damage to adjacent property by escaping water.

(D) *Ponding.*

41. The ponding system of curing may be used at the option of the contractor. All portions of the pavement surface shall be kept continuously covered with water for 10 days after the concrete is laid. Dikes shall be built to hold the water on the pavement.

(E) *Cleaning.*

42. The requirements for cleaning and for

(F) *Cold Weather Work.*

43. Cold weather work shall be as specified in Section X, (D) and (E) of Specifications for One Course Portland Cement Concrete Pavements for Highways.

XII. PROHIBITION OF TRAFFIC.

(A) *Barricades.*

44. The requirements for barricades shall be as specified in Section XI, (A) of the Specifications for One Course Portland Cement Concrete Pavements for Highways.

XIII. CONDITION FOR ACCEPTANCE.

45. Before the contract will be considered completed and the pavement ready for acceptance all equipment, surplus materials and construction debris of every description shall be removed from the streets and the pavement and parkings put in a neat and orderly condition. All manholes, catch-basins or other structures disturbed during construction shall be examined and any debris caused by the contractor shall be removed therefrom.

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Standard Specifications for Portland Cement Concrete Pavements.

Two-Course Portland Cement Concrete Street Pavement.*

I. GENERAL.

1. It is the intent of these specifications to cover the requirements for the materials and construction of Portland Cement Concrete Street Pavements composed of two layers of concrete made with unlike coarse aggregates, but of the same proportions.

II. MATERIALS.

2. The requirements for

(A) *Cement,*

(B) *Aggregates,*

(a) *Fine Aggregates,*

shall be as specified in Section II, (A), (B) and (a) of Specifications for One Course Portland Cement Concrete Pavements for Highways.

3. The requirements for

(b) *Coarse Aggregate for Bottom Course,*

(c) *Coarse Aggregate for Top Course,*

shall be as specified in Section II, (b) and (c) of (B) in Specifications for Two Course Portland Cement Concrete Pavements for Highways.

---

* Submitted by Committee S-6, on Concrete Roads and Pavements, for annual meeting, February, 1924. Accepted for Tentative Standard by meeting, February 26, 1924.

4. The requirements for

Crushed Rock,
Pebbles (Gravel),
Air Cooled Blast Furnace Slag,
Chatts or Tailings,

and for

(d) *Mixed Aggregates,*

(C) *Water,*
(D) *Reinforcement,*
(E) *Joint Filler,*

shall be as specified in Section II, of Specifications for One Course Portland Cement Concrete Pavement for Highways.

III. SUBGRADE.

5. The requirements for subgrade, including

(A) *Preparations and Maintenance of Subgrade,*
(B) *Checking and Acceptance of Subgrade,*
(C) *Aggregates on Subgrade,*
(D) *Special Treatment,*

shall be as specified in Section III, (A), (B), (C), (D) of Specifications for One Course Portland Cement Concrete Street Pavement.

IV. ADJUSTING STREET STRUCTURES.

6. Manhole and catch-basin covers, valve boxes and similar existing structures within the area to be paved shall be adjusted by the contractor to come flush with the pavement surface.

V. FORMS.

7. The requirements for

(A) *Materials,*

(a) *Wooden Forms,*
(b) *Metal Forms,*

(B) *Setting.*

shall be as specified in Section V, of Specifications for One Course Portland Cement Concrete Street Pavements.

VI. JOINTS.

8. The requirements for

(A) *General,*

(B) *Transverse Expansion Joints,*
(C) *Longitudinal Expansion Joints,*
(D) *Longitudinal Construction Joints,*
(E) *Manhole and Catch-basin Covers,*

shall be as specified in Section VI, (A) and (B), (C), (E), (F) of Specifications for One Course Portland Cement Concrete Street Pavements.

VII. WATER SUPPLY.

9. The requirements for

(A) *Water Taken from the City Mains,*
(B) *Priority to Water Supply,*

shall be as specified in Section VII, of Specifications for One Course Portland Cement Concrete Street Pavements.

VIII. PROPORTIONING AND MIXING CONCRETE.

10. The requirements for

(A) *Measuring Materials,*
(B) *Proportions,*

(a) *Proportioning Concrete With Mixed Aggregates,*

(C) *Mixing,*
(D) *Retempering,*
(E) *Central Mixing Plants,*
(F) *Consistency,*

shall be as specified in Section VIII, of Specifications for Two Course Portland Cement Concrete Pavements for Highways.

IX. PLACING CONCRETE AND REINFORCEMENT.

11. The requirements for

(A) *Inspection of Subgrade,*
(B) *Condition of Subgrade,*
(C) *Placing Reinforcement,*

shall be as specified in Section IX, (A), (B) of Specifiactions for Two Course Portland Cement Concrete Pavements for Highways.

(D) *Placing Concrete.*

12. The mixed concrete shall be deposited rapidly on the subgrade to the required depth and for the entire width between longitudinal joints, without the use of intermediate forms or bulkheads between joints. While being placed the concrete shall be vigorously sliced and spaded with suitable tools to eliminate voids or honeycomb pockets. The concrete shall be especially well spaded and tamped adjacent to forms, bulkheads and curbs. The bottom course shall be brought to the correct elevation and contour with a template or lute. The top course shall be placed within 15 min. after the bottom course. Any dust, dirt or foreign matter which collects on the surface of the bottom course shall be carefully removed before the top course is placed.

13. Whenever, because of a breakdown, or for any other reason, the placing of concrete will be stopped for more than 40 min., a transverse joint shall be formed at the point directed by the engineer, to close the section. Both the top and bottom courses shall be completed to this joint. Any concrete in excess of that needed to complete a section, when work is stopped for more than 40 min., shall not be used in the pavement.

X. FINISHING.

14. The requirements for

(A) *Leveling Surface*

shall be as specified in Section X, (A) of Specifications for One Course Portland Cement Concrete Street Pavements.

The requirements for

(B) *Belting,*
(C) *Longitudinal Floating,*
(D) *Finishing at Joints and Tooling,*
(E) *Trueness of Surface,*

shall be as specified in Section IX, paragraphs (D), (b), (d), (e), (f), of Specifications for One Course Portland Cement Concrete Pavement for Highways.

XI. CURING AND PROTECTION.

15. The requirements for

(A) *Burlap Cover,*
(B) *Wet Earth,*

shall be as specified in Section X, (A) and (B) of Specifications for One Course Portland Cement Concrete Pavement for Highways.

16. The requirements for

(C) *Sprinkling,*
(D) *Ponding,*
(E) *Cleaning,*
(F) *Cold Weather Work,*

shall be as specified in Section XI, (C), (D), (E), (F) of Specifications for One Course Portland Cement Concrete Street Pavement.

XII. PROHIBITION OF TRAFFIC.

17. The requirements for

(A) *Barricades*

shall be as specified in Section XI, of the Specifications for One Course Portland Cement Concrete Pavements for Highways.

XIII. CONDITION FOR ACCEPTANCE.

18. The requirements

shall be as specified in Section XIII, of Specifications for One Course Portland Cement Concrete Street Pavement.

DISCUSSION OF HIGHWAY AND PAVEMENT SPECIFICATIONS.

H. E. Breed (*by letter*).—The position of the American Concrete Institute is unique. So authoritative is its voice that whatever it approves stands as the last word in concrete construction. It is on this account that I beg the consideration of the Institute before it accepts as final the Specifications now before it for Concrete Highway Construction.

It is needless for me to expatiate here upon the importance of securing the best construction of our highways, or upon the part these specifications will play in such construction. My purpose is to urge that these specifications as printed be returned to the committee for further consideration in the light of more complete knowledge. Your committee, of which I am a member, has succeeded in holding but one meeting this past year, when at Atlantic City, we made the best rapid draft that we could, which, later put into commendable shape by Mr. Ege, is now in your hands. The value of the printing of this draft, as I see it, is that it may provoke discussion and comment. I should be loath to have it stand as our best opinion or as representing the mature judgment of the Institute. In another year I believe we shall be in a position to make an authentic report, including with it a revision of recommended practice, upon which the Institute may take definite action.

I believe that we shall be in a position to present an improvement over the specifications for the following reasons:

1. Some members of Committee of Aggregates are pursuing intensive research whose results, to be announced later, should be embodied in a final specification. Surface erosion and spalling of cracks seem frequent where certain classes of aggregates are used. Our specifications should tend to eliminate those aggregates. At present they do not accept in general terms without significance to the average engineer.

2. Test highways, especially the Bates Road, have shown the need for an interlocking joint. This year's work will ascertain the best form of such joint, which should be embodied in the specifications.

3. The sections on reinforcement or binding steel as they now stand are open to questions which further discussion might clarify and which this year's experiments and experience may answer.

4. The sections on subgrade can be greatly improved by incorporating into them whatever is valuable in the complete published reports of the Bureau of Public Roads, not available when we met, and of other bodies experimenting and reporting to the National Highway Research Council.

W. M. Acheson.—We propose to go on and draft a recommended practice which we hope to have in the hands of the Institute next year.

I believe the aggregates in the present specifications are as good as could be arrived at. They are the aggregates practically used in two, three or four of the large states, and I cannot see any reason why that request should receive any consideration. These highways, and especially

the Bates road, have shown the need of an interlocking joint. This year's work will ascertain the best type of such joints. I do not believe we should hold up the building of roads or the issuing of a set of specifications because something has not been developed which is a certainty.

E. E. HUGHES.—Last year I had an opportunity to go before the Road Committee and got there a day late, and I asked permission to present a little statement with regard to rail steel, which was permitted and was to go into the proceedings of the convention. My understanding was at that time that that statement should go before this committee. Unfortunately I have not been able to get in touch with the committee at this meeting. There is one question, however, that I would like to bring before the meeting.

The question involves the use, together with billet steel, of rail steel. They refer in this specification to the A. S. T. M. specification serial A 15-14 alone. The A. S. T. M. has another specification for rail steel serial designation A 16-14. I am perfectly frank to say that I know there is a good deal of prejudice among some engineers with regard to the use of rail steel, but if there is any one place where rail steel might be used, it certainly is in road building, and I feel that they do not specifically eliminate, but they refer specifically to this specification A 14-15, which does eliminate rail steel. I feel that if they would simply refer to the specifications of the A. S. T. M. A 15-14 and A 16-14, it would give this specification more value. From an economic standpoint, it would give bidders an opportunity to bid on the different grades of steel. If an engineer does not want to use it, he does not have to.

I feel that I have not had an opportunity to appear before this committee. I had a talk with Mr. Acheson after the matter was all closed up, and he was frank to say that he did not know very much about rail steel and he never had seen Prof. Hatt's report nor the Joint Committee's report on standard specifications, which in their metal reinforcement provision includes both grades of steel. It seems to me that if this is to be a standard specification, we have never had our day in court, we have never had an opportunity to appear before the committee and present our side of this matter. We would like to have the specification amended to include A. S. T. M. Specification A 16-14.

MR. ACHESON.—Mr. Hughes is right in what he says. I stated that as far as my general information connected with re-rolled steel was concerned, it was not worth much, but I told him we were guided absolutely by the committee, and in that committee at that time we took the rules governing the New York State Highway Department, the Pennsylvania Highway Department, and I think the Illinois Department, and the committee was unanimous in it, and our specifications provide for structural steel and intermediate grades. We did what we thought was the very best.

In this connection I will go a little further. Last year the proposition was brought to me for the use of re-rolled steel in our highways, and I took it up with the committee and recommended it for the reason that they

stated that they could get the bars. We wanted the highway to progress and that was very important, and they took it to the testing engineers at the Rensselaer Polytechnic Institute, and the report I got back on it was that it could not be done. That governed me largely in my attitude and vote on the committee.

There is only one other point I want to bring out. We had our meeting at Atlantic City. It was determined that we were to have another in Syracuse and that was arranged and the three members of the committee not here today were the three who wired and informed us that they could not arrange their affairs so they could be at that meeting. Now we get this eleventh hour letter asking that the specifications be held up for another year. Personally I am against it.

MR. HUGHES.—If a person has not had an opportunity to appear before the committee and present his side of the matter, it is a difficult thing to come here and offer an amendment to turn down the committee, but I thought there might be a possibility of referring this one provision back to the committee.

CHAIRMAN HUMPHREY.—A motion to that effect would be in order.

MR. HUGHES.—I will make that motion, and if it is adopted that will give us an opportunity to appear before the committee with proper supporting data and in that way broaden the scope of the work of the committee. (Motion seconded.)

CHAIRMAN HUMPHREY.—The question, as the Chair understands it, is, that in this particular paragraph (No. 17 in the major specification) there shall be an asterisk with a footnote which will state that the convention referred this paragraph back to the committee for further consideration.

The motion of Mr. Hughes was adopted by a majority vote.

AMERICAN CONCRETE INSTITUTE STANDARD.

Tentative Standard Specifications for Portland Cement Concrete Sidewalks.*

Submitted by Committee C-2 on Concrete Floor Finish.

I. ORIGIN AND USE.

(A) *Origin.*

Origin. These specifications are based on "Proposed Revised Specifications for Concrete Sidewalks," and subsequent revisions thereto included in the Reports* of the Committee on Sidewalks and Floors of the American Concrete Institute for 1918 and 1919. The specifications as printed herein were arranged by Committee C-2, on Concrete Floor Finish, and really constitute an entire rewriting of the specifications offered by the Committee on Sidewalks and Floors in 1918 and 1919.

(B) *Use of Specifications.*

Use. These specifications cover both one and two course sidewalk construction. The specifications for materials and construction included in paragraphs ——— to ——— and ——— to ——— inclusive, apply to both one and two course types of sidewalks. The use of the specifications will be facilitated by the following subdivision of them by paragraphs according to the type of sidewalk involved:

For one course sidewalks use Paragraphs

For two course sidewalks use Paragraphs

II. MATERIALS.

General. 1. Before delivery upon the job, and at such other times as the engineer deems necessary, the contractor shall furnish any required samples of the materials hereafter mentioned. Materials shall pass the following requirements.

(A) *Cement.*

Cement. 2. The cement shall meet the requirements of the current Standard Specifications for portland cement of the American Society for Testing Materials. A sack containing 94 pounds of cement will be considered one cubic foot.

---

* Printed in Proceedings A. C. I., Vol. XIV, p. 489, with revisions printed in Proceedings A. C. I., Vol. XV, p. 413.

Adopted as Tentative Standard, Annual Convention, February 26, 1924.

(B) *Fine Aggregate.*

3. The fine aggregate shall consist of clean, hard, durable, uncoated particles of sand or stone, free from all organic material. 100 per cent shall pass a ¼ in. screen and 95 per cent shall be retained on a 100-mesh screen. Not more than 25 per cent shall pass a 50-mesh screen. It shall be well graded from coarse to fine, and shall contain not more than 5 per cent by weight of clay or loam, none of which shall be in lumps. **Fine Aggregate.**

4. When the fine aggregate is mixed with portland cement in the proportion of one part cement to three parts fine aggregate by weight, according to the standard method of making Compression Tests of Concrete of the American Society for Testing Materials, the resulting mortar at the age of 7 and 28 days shall have a compressive strength of at least ——— and ——— pounds per square inch respectively. **Permissive Test.**

(C) *Coarse Aggregate.*

5. Coarse aggregate may be broken stone, gravel or blast furnace slag. 100 per cent of the coarse aggregate shall pass a one inch screen and at least 95 per cent shall be retained on a ¼ in. screen, with all intermediate sizes retained. **General.**

6. The broken stone or gravel shall be clean, hard, durable, uncoated rock. It shall contain no vegetable or other deleterious matter and shall be practically free from soft, thin, elongated or laminated pieces. **Broken Stone or Gravel.**

7. Slag shall be of a quality which has been successfully used in local concrete work. It shall comply with the A. S. T. M. Specification C 33-23 T. **Slag.**

(D) *Water.*

8. Water shall be clean and practically free from alkali, oils or acid.

(E) *Joint Filler.*

9. The joint filler shall be a suitable elastic waterproof compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather, or prepared strips of fiber matrix and bitumen as approved by the engineer. The strips shall be ½ in. in thickness, their width shall at least equal the full thickness of the slab and their length shall at least equal the width of the slab at the joint. **Joint Filler.**

(F) *Forms.*

10. Forms shall be of lumber two inches thick, or of steel of equal strength, except on curves, where flexible strips may be used. **General.**

(G) *Division Plates.*

11. Where division plates are used they shall be of ⅛ in. steel, as wide as the depth of the slab and as long as the width of the walk.

(H) *Subbase.*

Subbase.

12. Only clean, durable material, such as coarse gravel or steam-boiler cinders free from ash or particles of unburned coal shall be used in the subbase. (Note.—Eliminate this clause unless subbase is required.)

III. CONSTRUCTION.

(A) *Subgrade.*

Definition.

13. That portion of the ground surface directly beneath the slab shall be called the subgrade.

Preparation of Subgrade.

14. All soft and spongy material in the subgrade shall be removed and replaced with suitable material. Fills shall be compacted in layers not exceeding six inches in thickness. Spots previously compacted by traffic shall be loosened to a depth of six inches. The whole subgrade shall be thoroughly and uniformly compacted to a firm surface having as nearly as possible a uniform bearing power.

Surface of Subgrade.

15. A template, resting upon the side forms and having its lower edge at the exact elevation of the subgrade, shall be drawn along the forms before any concrete is laid. Any high places in the subgrade shall be removed so that the template will pass over without being raised off the side forms or being tipped at an angle to the sidewalk surface.

Wetting.

16. The subgrade shall be damp, but not muddy, when concrete is placed upon it.

(B) *Drains.*

17. Where the nature of the subgrade is such as to produce unusual frost action, drains of 4 inch concrete tile shall be laid on the lines and grades given by the engineer.

(C) *Subbase.*

Applying Material

18. The subbase material shall be spread, thoroughly rolled and tamped to a surface at least ——— in. below the finished grade of the walk. On fills, the subbase shall have the same slope as the sides of the fill.

Wetting.

19. While compacting the subbase, the material shall be kept thoroughly wet, and shall be wet when the concrete is deposited, but shall show no pools of water.

(D) *Forms.*

Forms.

20. Forms shall be held rigidly in place by stakes or braces with top edges at true line and grade, to give the walk a slope toward the curb of ¼ in. per foot of width. Ends of adjoining forms shall be flush.

Cleaning and Oiling.

21. Forms and division plates shall be thoroughly cleaned and oiled each time before they are used.

(E) *Measuring and Mixing.*

22. The method of measuring the materials for the concrete or mortar, including water, shall be one which will insure separate and uniform proportions of each of the materials at all times. A sack of portland cement (94 lb. net) shall be considered one cubic foot. Cement shipped in bulk shall be proportioned by weight. **Measuring.**

23. All concrete shall be mixed by machine except when the architect or engineer shall otherwise permit under special conditions. A batch mixer of an approved type shall be used. The ingredients of the concrete or mortar shall be mixed to the specified consistency, and the mixing shall continue for at least one minute after all materials are in the drum. The drum shall be completely emptied before receiving material for the succeeding batch. **Machine Mixing.**

24. When it is necessary to mix by hand, the materials shall be mixed dry on a watertight platform until the mixture is of uniform color, the required amount of water added, and the mixing continued until the mass is of uniform consistency and homogeneous. **Hand Mixing.**

25. Retempering of mortar or concrete which has partially hardened, that is, remixing with or without additional materials or water, shall not be permitted. **Retempering.**

26. The consistency of the mixed concrete shall be such that no separation of the ingredients takes place and some tamping is necessary to bring the water to the surface. **Consistency.**

(F) *Cold Weather Work.*

27. Concrete shall not be placed on a frozen subgrade or when the temperature is, or is liable to be within 24 hours, below 35 degrees Fahrenheit, except with the written permission of the engineer, and according to his instructions. Freshly placed concrete shall not be allowed to freeze for a period of five days after placing. **Cold Weather Work.**

(G) *Jointing.*

28. The walk shall be cut into separate rectangular slabs. No plain concrete slab shall be longer than six feet on any one side. **Construction Joints.**

29. Division plates shall be removed after the concrete has hardened sufficiently to avoid breaking the edges or corners of the slabs. **Division Plates.**

30. Where division plates have not been used, the partially set concrete shall be cut through to the subgrade or subbase. Care shall be taken to make the cut at right angles to the surface of the sidewalk. **Cut Joints.**

31. The surface edges of each slab shall be rounded to a radius of about ¼ inch. Markings shall be exactly at cuts between slabs. **Edges.**

(H) *Expansion Joints.*

General.

32. Expansion joints shall extend from the surface to the subgrade, be truly at right angles to the sidewalk surface and be made by putting the specified joint filler in place before placing the concrete. They shall be placed as follows:

At Curb Line.

(a) 33. At or near all places where the sidewalk line intersects a curb line or other sidewalk a one inch expansion joint shall be made at right angles to the center line of the walk.

Longitudinal Expansion Joints.

(b) 34. When the sidewalk fills the space between the curb and the building line a ½ in. expansion joint shall be placed between the curb and the sidewalk, and between the sidewalk and the building.

Cross Joints.

(c) 35. A ½ in. expansion joint shall be made across the walk at approximately 50-ft. intervals.

(I) *Curing and Protection.*

36. As soon as the concrete has set sufficiently it shall be sprinkled and kept moist until covered. As soon as it can be done without damage to the walk it shall be covered with two inches of earth or sand which shall be kept wet seven days. The walk shall then be cleaned and opened to traffic.

37. The contractor shall protect the concrete from damage by rain, pedestrians and animals, with suitable covers and barricades, and by red lights at night.

IV. ONE COURSE WORK.

(A) *Thickness and Proportions.*

Thickness and Proportions.

38. The sidewalk shall consist of one 5 in. course of concrete in the proportion of one part of portland cement, two parts of fine aggregate and three parts of coarse aggregate.

(B) *Placing and Finishing.*

Placing.

39. The freshly mixed concrete shall be placed immediately on the prepared subgrade. It shall then be struck off and tamped with a straightedge resting upon the side forms and advanced with a crosswise sawing motion.

Finishing.

40. It shall then be floated with a wooden float until the surface has a true surface and the concrete is thoroughly compacted.

V. TWO COURSE CONSTRUCTION.

41. The above specifications shall govern the construction of two-course sidewalks except Paragraph 38, "Thickness and Proportions," and Paragraphs 39 and 40, "Placing" and "Finishing," which shall be replaced by the following:

(A) *Thickness Proportions.*

42. Two course sidewalks shall consist of a base 4¼ in. thick composed of concrete in the proportions one part portland cement, three parts of fine aggregate and five parts coarse aggregate and a top coat ¾ in. thick composed of mortar in the proportions one part portland cement and two parts fine aggregate. **Thickness and Proportions.**

(B) *Base.*

43. The base shall be deposited on the subgrade and thoroughly compacted by tamping. It shall then be struck off by a template which shall leave it nowhere less than ¾ of an inch below the finished surface. **Laying Base.**

(C) *Wearing Surface.*

44. Within 45 minutes after the bottom course is laid and before the initial set has taken place, the material for the wearing surface shall be placed and brought to the established grade by means of a strike board. **Laying.**

(D) *Finishing.*

45. After the wearing course has been brought to the established grade it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. **Finishing.**

N. M. LONEY, *Chairman.*

A. C. IRWIN, *Secretary.*

AMERICAN CONCRETE INSTITUTE STANDARD.

STANDARD SPECIFICATIONS FOR CONCRETE FLOORS.*

I. Application and Use

A. FOREWORD

These "Standard Specifications for Concrete Floors" of the American Concrete Institute, with supplementary notes explaining and amplifying the specifications were prepared by the Committee on Concrete Floor Finish. The specifications are printed on the left hand pages and the notes on the right hand pages, in order to facilitate cross reference.

B. APPLICATION OF SPECIFICATIONS

These specifications apply to floors in buildings, whether subjected to moderate or heavy traffic, and cover the laying and finishing of the floor; also its protection during early hardening.

C. USE OF SPECIFICATIONS

For architects, engineers and others desiring to embody these specifications in their general specifications covering a particular piece of work, the following outline of the paragraphs necessary to meet different conditions will prove convenient:

Floors Laid on Ground:—Moderate or Light Traffic.
Two-Course—Paragraphs 1-19 (except 8); 34-52; 54-57.
One-Course—Paragraphs 1-19 (except 8); 34-47; 58-62.

Floors Laid on Ground:—Heavy Traffic.
Two-Course—Paragraphs 1-19; 34-51; 53-57.

Reinforced-Concrete Floors:—Moderate or Light Traffic,
Paragraphs 1-26 (except 8); 28-33.

Reinforced-Concrete Floors:—Heavy Traffic: Paragraphs 1-25; 27-33.

---

* As printed in Proc., A.C.I., vol. XIV, 1918, p. 496, with revisions printed in Proc. A.C.I., vol. XV, 1919, p. 413, and as revised by Committee C-2 on Concrete Floor Finish 1922. Accepted by Annual Meeting, January, 1923. Adopted as Standard by letter ballot April 29, 1924.

II. MATERIALS

A. PORTLAND CEMENT

Portland Cement

1. Portland cement shall meet the requirements of the current Standard Specifications for Portland Cement adopted by the American Society for Testing Materials.

B. AGGREGATES

General Requirements

2. Before delivery on the job, the contractor shall submit to the architect or engineer a fifty (50) lb. sample of each of the aggregates proposed for use. These samples shall be tested, and if found to pass the requirements of the specifications, similar material shall be considered as acceptable for the work. In no case shall aggregate containing frost or lumps of frozen material be used.

Crusher run stone, bank run gravel or mixtures of fine and coarse aggregate prepared before delivery on the work shall not be used.

C. FINE AGGREGATE

General Requirements

3. Fine aggregate shall consist of natural sand or screenings from hard, tough crushed rock or gravel consisting of quartz grains or other hard material, clean and free from any surface film or coating and graded from fine to coarse, with the coarse particles predominating.

Grading

4. Fine aggregate, when dry, shall pass a screen having four (4) meshes to the linear inch; not more than twenty-five (25) per cent shall pass a sieve having fifty (50) meshes per linear inch and not more than five (5) per cent shall pass a sieve having one hundred (100) meshes per linear inch.

Impurities

5. Fine aggregate shall not contain injurious vegetable or other organic matter as determined by the colorimetric test nor more than five (5) per cent by volume of clay or loam. Field tests may be made by the architect or engineer on fine aggregate as delivered at any time during the progress of the work. If there is more than seven (7) per cent of clay or loam by volume in one (1) hour's settlement after shaking in one hundred (100) per cent excess of water, the material represented by the sample shall be rejected.

Mortar Strength Test

6. Fine aggregate shall be of such quality that mortar composed of one (1) part portland cement and three (3) parts fine aggregate, by weight, when made into briquets, shall show a tensile strength at seven (7) and twenty-eight (28) days at least equal to the strength of briquets composed of one (1) part of the same cement and three (3) parts standard Ottawa sand, by weight. The percentage of water used in making the briquets of cement and fine aggregate shall be such as to produce a mortar of the same consistency as that of the Ottawa sand briquets of standard consistency. In other respects all briquets shall be made in accordance with the methods of testing cement recommended by the American Society for Testing Materials. (See Standard Specifications and Tests for Portland Cement of the A. S. T. M. Serial Designation C 9-21.)

II. MATERIALS

Aggregates. Many kinds of stone, from very hard quartzite to soft limestone, are crushed to the required sizes and used as concrete aggregates. Similarly, many varieties of pebbles are used. The natural tendency is to use materials at hand; but if aggregates are not of good quality or if they contain too much clay or other impurities, a poor concrete will result. Hence it is highly important that the aggregates be tested unless their quality is known.

The use of frozen aggregates results in very inferior concrete. Lumps of ice and frozen gravel go through the mixer and into the forms without being combined with the cement. When they thaw out, voids or stone pockets are left in the concrete.

Aggregates are usually considered as belonging to one of two classes: Fine aggregate, composed of sand or stone screenings, and coarse aggregate, composed of crushed stone or pebbles, the line of division, purely an arbitrary one, being marked by the screen having ¼-in. meshes. This division provides a simple and customary means of properly proportioning the aggregate as a whole.

Unwashed materials may contain clay or organic impurities, comparatively small amounts of which may have ruinous effects on concrete. More than seven (7) per cent of clay by volume may cause a serious loss of strength of the concrete, while even smaller amounts if present as a surface film on the particles, reduce the strength of concrete by preventing proper bonding with the cement. A recent series of tests at the Structural Materials Research Laboratory showed that the presence of organic matter to the extent of only 1/1000 part of the weight of the fine aggregate may decrease the strength of the resulting concrete by twenty-five (25) per cent. The field test for clay or loam and the colorimetric test for organic matter are simple methods for determining the presence of such impurities.

The colorimetric test may be applied in the field as follows: Fill a twelve (12) ounce graduated prescription bottle to the four and one-half (4½) ounce mark with the sand to be tested. Add a three (3) per cent solution of sodium hydroxide until the volume of sand and solution, after shaking, amounts to seven (7) ounces. Shake thoroughly and let stand for twenty-four (24) hours. The sample shall then show a practically colorless solution, or at most a solution not darker than straw color.

The hardness of the aggregate is a feature to be considered, but it is of lesser importance in influencing the strength or resistance to abrasion of concrete, than the methods of mixing and placing and of protecting the floor during the early hardening period. However, where great resistance to abrasion is essential, as in the construction of floors that will be subjected to heavy trucking, the hardness of the aggregate (particularly for the wearing course, if there is one) becomes most important, and in such cases the use of small pebbles or small pieces of hard crushed rock (No. 1 aggregate for wearing course) is desirable. If used in the proportions given in paragraph 27, such aggregate will help to produce a harder and more wear-resistant surface.

To secure good concrete the coarse aggregate should have a maximum size of approximately two (2) inches for plain concrete and 1 in. for reinforced work. Slabs less than four (4) inches thick

D. COARSE AGGREGATE

Coarse Aggregate

7. Coarse aggregate shall consist of clean, hard, tough, crushed rock or pebbles graded in size, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. The size of the coarse aggregate shall range from one and one-half (1½) in. down, not more than five (5) per cent passing a screen having four (4) meshes per linear inch, and no intermediate sizes shall be removed.

No. 1 Aggregate

8. No. 1 aggregate for the wearing course shall consist of clean, hard, tough, crushed rock or pebbles, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. It shall pass when dry a screen having three-eighths (⅜) in. openings and not more than ten (10) per cent shall pass a screen having four (4) meshes per linear inch.

E. WATER

General Requirements

9. Water shall be clean, free from oil, acid alkali or vegetable matter.

F. COLOR

General Requirements

10. If artificial coloring matter is required, only those mineral colors shall be used which, in the amount hereinafter specified, will not appreciably impair the strength of the cement.

G. REINFORCEMENT

Specifications and General Requirements

11. The reinforcing metal shall meet the requirements of the current Standard Specifications for Steel Reinforcement of the American Society for Testing Materials. It shall be free from excessive rust, scale, paint or coatings of any character which will tend to reduce or destroy the bond.

H. JOINT FILLER

Joint Filler

12. The joint filler shall be a suitable compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather; or, prepared strips of fiber matrix and bitumen as approved by the architect or engineer. The strips shall be one-half (½) in. in thickness and their width shall at least equal the full thickness of the slab.

III. CONSTRUCTION

A. PROPORTIONING

Method of Measuring

13. The method of measuring the materials for the concrete or mortar, including water, shall be one which will insure separate and uniform proportions of each of the materials at all times. A sack of portland cement (94 lb. net) shall be considered as one (1) cu. ft.

and heavily reinforced may require ¾ in. as a maximum. A well graded coarse aggregate generally produces a denser and stronger concrete than an aggregate of uniform size.

Mixed Aggregate. Sand and pebbles from a pit, or crushed stone from a crusher are seldom if ever properly graded for use in concrete. There is nearly always a surplus of fine material from a pit and of coarse material from a crusher. That is, if one hundred (100) cu. ft. of material from a gravel pit is passed over a one-fourth (¼) in. screen, perhaps seventy (70) cu. ft. will pass through and forty (40) cu. ft. will be retained on the screen. If a proper gradation is obtained by the use of two (2) cu. ft. of fine with four (4) cu. ft. of coarse material, only 20 cu. ft. of the fine material should be used with the forty (40) cu. ft. of pebbles retained on the screen. If the bank-run material were used, the resulting proportion would be seven (7) to four (4) instead of two (2) to four (4). This is the fundamental objection to the use of bank-run material, and a similar objection applies to crusher run material.

Prepared aggregate mixtures, which are sometimes supplied in carload lots, develop the same objections because coarse and fine materials become somewhat separated in transit so that no two individual wagon loads taken from the car will have the same relative proportions of fine and coarse aggregate. It is far better to handle the fine and coarse materials separately and combine them in proper amounts *at the mixer.*

Water. A safe rule for mixing concrete is to use only water which is fit to drink.

Color. Mineral coloring material is preferred to organic coloring material, because the latter fades more than mineral colors, and because it may seriously reduce the strength of the concrete. Mineral coloring may reduce the strength of concrete somewhat but where the quantities used are less than 5 per cent, this is not serious. The use of colored aggregates is preferable in obtaining color effects, the surface of the floor being brushed or ground to expose the aggregate.

MEASURING AND MIXING

Proportioning. Uniform proportioning, and uniform consistency of concrete are essential for high grade floor construction. If wheelbarrows are used to measure the aggregates, a simple method should be provided to check occasionally the amounts in the barrows. Boxes made to set in the barrows and holding one or more cubic feet are useful. Some barrows are built with level tops so that they may be struck off to known capacity. A small tank capable of delivering a fixed volume of water for each batch insures a more nearly uniform consistency than can be obtained when water is added by means of a pail. Batch mixers are now generally equipped with water measuring devices.

B. MIXING

Machine Mixing 14. All concrete shall be mixed by machine except when the architect or engineer shall otherwise permit under special conditions. A batch mixer of an approved type shall be used. The ingredients of the concrete or mortar shall be mixed to the specified consistency, and the mixing shall continue for at least one (1) minute after all the materials are in the drum. Raw materials shall not be permitted to enter the drum until all the material of the preceding batch has been discharged.

Hand Mixing 15. When it is necessary to mix by hand, the materials shall be mixed dry on a watertight platform, until the mixture is of uniform color, the required amount of water added, and the mixing continued until the mass is of uniform consistency and homogeneous.

Retempering 16. Retempering of mortar or concrete which has partially hardened, that is, mixing with or without additional materials or water, shall not be permitted.

C. CURING

Covering 17. As soon as the finished floor has hardened sufficiently to prevent damage thereby, the floor shall be covered with at least one (1) in. of wet sand, or two (2) in. of sawdust, which shall be kept wet by sprinkling with water for at least ten (10) days.

Protection 18. The freshly finished floor shall be protected from hot sun and drying winds until it can be sprinkled and covered as above specified. The concrete surface must not be damaged or pitted by raindrops, and the contractor shall provide and use when necessary sufficient tarpaulins to completely cover all sections that have been placed within the preceding twelve (12) hours.

D. TEMPERATURES BELOW 40° F.

Temperature below 40 Degrees Fahrenheit 19. If at any time during the progress of the work the temperature is, or in the opinion of the architect or engineer will, within twenty-four (24) hours, drop to 40 degrees Fahrenheit, the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least five (5) days.

Machine Mixing. Concrete must be thoroughly mixed before it is deposited in the forms. Experiments have shown that the strength of concrete increases rapidly with the time of mixing up to one minute. The rate at which the mixer revolves i. e. between 12 and 25 R. P. M. has little influence on the strength of concrete. A small mixer should not be speeded up where it is necessary to place a large volume of concrete in a short time, because insufficient mixing decreases the strength of concrete. If a given mixer cannot mix enough concrete thoroughly in the available time, a second mixer or a larger one should be provided. Continuous mixers do not produce as uniform concrete as do batch mixers and are therefore not recommended.

Hand Mixing. Where large volumes of concrete are to be placed, hand mixing will generally be found more expensive than machine, but good results may be obtained with hand mixing if the specifications are carefully followed.

Retempering. Concrete or mortar should be deposited in place as soon as possible after mixing; otherwise it becomes partially hardened. If that happens it should be thrown out because it will not attain its full strength even if remixed with other materials.

The hardening of concrete is a process which requires time and the presence of moisture. The more thorough the process the harder and stronger the concrete. As mentioned hereafter, it is important that a minimum amount of water be used in mixing the concrete, but after the concrete is deposited in place, it is even more important that its water content be retained and not allowed to drain off or evaporate. Drenching the subbase with water before depositing the concrete reduces the loss of water from below by gravity or capillary action, but that alone is not sufficient: Evaporation from above must also be prevented. A common method of protecting the concrete so as to retain its water content is to cover the surface with damp earth or sawdust as soon as it has hardened sufficiently to prevent injury thereby, and then to keep the sand damp by frequent sprinkling. Where feasible, an excellent method is to build small dams of clay or other suitable material around the floor, and then flood it with a few inches of water. In other cases a covering of *wet* burlap has been found satisfactory. This protection should be continued for at least ten days, and if possible for three weeks. Laboratory tests have shown that protecting the surface of a concrete floor in this manner for the first ten days will increase the compressive strength and resistance to wear fifty (50) per cent. ("Proceedings," American Concrete Institute, 1921, p. 251.) In other words, this one item of ten (10) days' protection will give the owner of a concrete floor fifty (50) per cent greater value for his money. Still better results will be obtained if this protection can be continued for three weeks, or longer if practicable.

Temperature Below 40° F. If concrete is allowed to freeze during the early hardening period, it may be seriously damaged and its hardening will be greatly delayed. Warmth as well as moisture is necessary for the proper hardening of concrete, so that the concrete should have a temperature of at least 60° F. when deposited and provision made to maintain this temperature for at least 5 days.

IV. Reinforced-Concrete Floors

For reinforced-concrete floors the following will apply in addition to paragraphs 1 to 19 incl.

A. FORMS

Forms 20. The forms shall be substantial, unyielding and so constructed that the concrete will conform to the designed dimensions and contours, and shall also be tight to prevent the leakage of mortar. The supports for floors shall not be removed until the concrete has hardened sufficiently and then only with the consent of the engineer or architect in charge. Permanent shores shall be placed in such a manner as to assure safety of the floors after temporary supports are removed.

B. REINFORCEMENT

Reinforcement 21. Reinforcing metal shall be provided as called for on the plans. It shall be placed as indicated and mechanically held in position so that it will not become disarranged during the depositing of the concrete. Whenever it is necessary to splice tension reinforcement, the character of the splice shall be such as will develop its full strength. Splices at points of maximum stress shall be avoided. Splicing by lapping bars without contact and with space between bars along the overlap equal to twice the thickness of the bars is preferable to mechanical splices or clamps.

C. CONCRETE SLAB

Proportions 22. The concrete shall be mixed in the proportions by volume of one (1) sack of portland cement, two (2) cu. ft. of fine aggregate and four (4) cu. ft. of coarse aggregate.

Consistency 23. Only sufficient water shall be used to produce a workable plastic mix, which will flow sluggishly into the forms and around the reinforcement and which can be conveyed from the mixer to the forms without the separation of the coarse aggregate from the mortar.

Placing 24. The concrete shall be placed in a manner to insure a smooth ceiling, and thoroughly worked around the reinforcement and into the recesses of the forms. Concrete shall be deposited in its final position as soon as possible after mixing. It shall be struck off to a surface at least one (1) in. below the established grade of the finished surface of the floor. Workmen shall not be permitted to walk in freshly-laid concrete, and if sand or dust collects on the base, it shall be carefully removed before the wearing course is applied.

Joints 25. When it is necessary to make a joint in a floor slab, its location shall be designated by the architect or engineer; joints to be vertical.

REINFORCED-CONCRETE FLOORS

Forms. When forms carrying concrete floors sag out of place before the concrete has hardened it is very difficult to force them back into position. Therefore, care should be taken before placing is commenced to have the forms and false work of such strength that there is no danger of sagging or failure.

The weight of a reinforced concrete floor is sometimes nearly equal to the load it is expected to carry. If sufficient time is not allowed for hardening there is possibility of failure, whereas an excellent floor would have resulted had it been allowed to harden properly before the supports were removed. This is especially true in cold weather, because concrete, without proper protection hardens very slowly if at all when the temperature is below 40 degrees. Hence it is safe to assume that a concrete floor gains practically no strength during the time exposed to low temperature.

Reinforcement. The value of reinforcing steel depends upon position as well as quantity of steel. The designed strength of a floor is based on the assumption that the steel will be in the position shown on the plans. If the reinforcing is out of place its value will be decreased, or it may be subjected to possible corrosion because of insufficient protecting concrete below. A three-fourth (¾) inch covering should be the minimum and should be increased as conditions may require.

CONCRETE SLAB

Consistency. Few engineers and contractors realize the damage caused by the use of too much water in mixing concrete. The statement is frequently made that excess mixing water does no harm because it soon runs off or evaporates, and that very wet concrete gains strength more rapidly than dry concrete, but this is not correct. A series of tests made at the Structural Materials Research Labratory, Chicago, shows that the quantity of mixing water is probably the most important factor affecting the strength and durability of concrete, and that down to a point lower than can be reached in ordinary concrete work, the smaller the quantity of mixing water, the stronger will be the concrete. The use of one (1) pint more water than necessary in a one (1) bag batch decreases the strength and resistance to wear of the resulting concrete as much as though two (2) or three (3) pounds of cement were left out. Concrete of sloppy consistency has less than half the strength of concrete of the same proportions mixed with the proper amount of water. Therefore the best rule is to use the minimum quantity of water that will produce a workable, plastic, mix.

For reinforced concrete a somewhat wetter consistency is necessary in order that the mixture will settle readily around the reinforcing bars and fill all the spaces between them, but considerably less water should be used than is customary, and much better concrete would then result.

The slump test is a simple method for determining the proper consistency for the work in hand. A frustum of a cone, four (4) ins. in diameter at the top, 8 inches at the bottom and twelve (12) ins. high, made of sheet metal, is filled with the mixture to be tested, the concrete being puddled with a pointed metal rod while the cone is being filled. The cone is immediately lifted off and the settlement or slump noted. For a plain concrete floor slab the proper slump is one (1) in. to one and one-half (1½) to one (1) in., and for a reinforced-concrete floor slab, two (2) in. to two and one-half (2½) in. A greater slump indicates the use of too much water. In some cases where the reinforcement is complicated, the strength may be retained by adding a sufficient amount of cement to keep the water-cement ratio unchanged.

Joints. Floor finish should not be divided or scored or blocked off by scoring tools except at structural expansion joints as this leads to rapid wear when subject to impact of traffic wheels.

D. WEARING COURSE

Proportions and Thickness

26. (Mixture No. 1) The mortar shall be mixed in the proportions of one (1) sack of portland cement, and two (2) cu. ft. of fine aggregate. The minimum thickness shall be three-quarters (¾) in.

27. (Mixture No. 2) The mortar shall be mixed in the proportions of one (1) sack of portland cement, one (1) cu. ft. of fine aggregate and one (1) cu. ft. of No. 1 aggregate for wearing course. The minimum thickness shall be three-quarters (¾) inch.

Consistency

28. The mortar shall be of the dryest consistency possible to work with a sawing motion of the strikeboard.

Placing

29. The wearing course shall be placed immediately after mixing. It shall be deposited on the fresh concrete of the base before the latter has appreciably hardened, and brought to the established grade with a strikeboard.

Note. When placing the wearing course after the concrete slab has hardened, eliminate paragraph 29 and substitute paragraphs 30 and 31.

Preparation of Slab

30. The surface of the slab shall be thoroughly roughened by picking or other means and cleaned of all dirt and debris.

Placing

31. The slab shall be thoroughly moist but free from pools of water when the grout and mortar for wearing course is placed. A neat cement grout shall be brushed on the surface of the slab, the wearing course immediately applied and brought to the established grade with a strikeboard. Grout and mortar shall be used within forty-five (45) minutes after mixing with water.

Finishing

32. After the wearing course has been brought to the established grade by means of a strikeboard, it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. When required, the surface shall be steel-troweled, but excessive working shall be avoided. A mixture of dry cement, sand and number one aggregate may be applied to the fresh concrete of the base for a wearing course, but in no case shall dry cement or a mixture of dry cement and sand be sprinkled on the surface of the wearing course to absorb moisture or to hasten the hardening. Special methods not conflicting with these specifications may be used.

Coloring

33. If artificial coloring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform color. In no case shall the amount of coloring exceed five (5) per cent of the weight of the cement.

WEARING COURSE

Proportions and Thickness. Either of the two (2) mixes specified insures an excellent finish. As indicated in notes on aggregates, however, the No. 2 mixture using small pebbles or stone chips with sand resists abrasion better than No. 1 mixtures and is preferred where the floor will be subjected to heavy traffic.

Consistency. The above remarks on the consistency of the concrete for the slab apply with equal or added force to the wearing course, for this is the part of the floor which must withstand all the abrasive action of traffic.

Placing. Where possible, the wearing course should be placed before the base course has hardened appreciably, because this insures a good bond and a practically monolithic floor. In case this is not possible, or when it is necessary to renew the wearing course on an old floor, the precautions given in paragraphs 30 and 31 should be carefully observed. Failure to do so may cause a poor bond, which may allow the wearing course to work loose from the base, crack and break up under traffic. Roughening the slab gives a mechanical bond. The slab should obviously be free from dirt and refuse. In order to reduce absorption of water from the fresh wearing course, which would prevent it from hardening properly, the slab should be moist, but not covered by a film of water, which might affect the bond. A grout of neat cement painted or brushed into the surface of the slab just before the wearing course is placed insures a thorough bond. Tests made by the Bureau of Standards have shown this to be more effective than special treatments. If these precautions are followed and the surface protected properly while hardening, a satisfactory, wear resistant floor will be secured.

Finishing. Working the surface of a concrete floor with a wood float smooths out any inequalities and compacts the surface without drawing to the top the finer particles of cement and sand. All of this adds to the value of the floor. Working with a steel trowel gives a smoother finish, but excessive troweling tends to bring fine particles in the mixture to the surface. These fine particles are not firmly cemented together and loosen rapidly under traffic, thus causing objectionable dust. The same objectionable feature results from sprinkling dry cement or a dry mixture of cement and sand on the finished surface.

In all cases, as soon as the floor has hardened sufficiently, it should be protected from too rapid drying by a covering of damp sand or by flooding with water.

Reference to special methods will permit the use of the so-called "monolithic method" of finishing concrete floors in buildings, but prohibits the practice of drying up excess water on the surface of a wearing course by dusting on a drier.

Coloring. See previous remarks on page 493.

V. Plain Concrete Floors

For plain concrete floors the following will apply in addition to paragraphs 1 to 29 incl.

A. SUBGRADE

Preparation 34. All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding six (6) in. in thickness. The subgrade shall be thoroughly tamped until it is brought to a firm, unyielding surface.

Deep Fills 35. All fills shall be made in a manner satisfactory to the architect or engineer. The use of muck, quicksand, soft clay, spongy or perishable material is prohibited.

Drainage 36. When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the engineer.

Depth 37. The subgrade shall not be less than ——— in. below the finished surface of the floor.

Note. Subgrade is to be —— in. below the finished surface of the floor when subbase is not required, and at least —— in. below when subbase is required.

B. SUBBASE

(Omit these sections when subbase is not required.)

Subbase 38. Only clean, hard material, such as coarse gravel or steamboiler cinders, free from ash or particles of unburned coal, shall be used in the subbase.

Thickness 39. The material as specified shall be spread on the subgrade thoroughly rolled or tamped to a surface at least —— in. below the finished grade of the floor. On fills, the subbase shall extend the full width of the fill.

Wetting 40. While compacting the subbase, the material shall be kept thoroughly wet, and shall be in that condition when the concrete is deposited.

C. FORMS

Materials 41. Forms shall be free from warp and of sufficient strength to resist springing out of shape.

Setting 42. The forms shall be well staked or otherwise held to the established lines and grades and their upper edges shall conform to the established grade of the floor.

Treatment 43. All wood forms shall be thoroughly wetted and metal forms oiled or coated with soft soap or whitewash before depositing any material against them. All mortar and dirt shall be removed from the forms that have been previously used.

Subgrade. Where concrete floors are laid directly on the ground, it must be firm and unyielding in order to support the floor properly. Spongy places cause the floor to settle unevenly and crack. If natural drainage is poor so that the subgrade may become saturated with water, a suitable drainage system should be provided, not only to prevent settlement, but also, in the case of basements, to insure watertightness.

If the subgrade is of sand or gravel the floor can be laid directly upon it; otherwise a layer of coarse gravel or steam boiler cinders at least six (6) inches thick should be spread upon the subgrade and thoroughly compacted by wetting and rolling or tamping. A coarse, durable material is obviously necessary for this purpose.

As mentioned on page 11 the subbase should be wet when the concrete is deposited, in order to minimize loss of water from the concrete by absorption.

Forms. Forms for concrete, whether wood or metal, are more easily stripped and leave a smoother finish if they are coated with paraffin or some other oil before the concrete is deposited. Soft soap or whitewash answer the same purpose. For further discussion see page 9.

D. LIMITING CONDITIONS

Size of Slabs

44. The slabs or independently-divided blocks when not reinforced shall have an area of not more than one hundred (100) sq. ft., and shall not have dimensions greater than ten (10) ft. Larger slabs shall be reinforced as hereinafter provided.

Thickness of Floor

45. The thickness of the floor shall be not less than —— inches.

Width and Location of Joints

46. When required by the architect or engineer in charge, a one-half (½) inch space or joint shall be left between the floor and the walls and columns of the building, to be filled with the material before specified under "Joint Filler."

Protection of Edges

47. Where required by the architect or engineer in charge, the edges of the slabs at the joints shall be protected by metal. Unless protected by metal, the upper edges of the slabs shall be rounded to a radius of one-half (½) inch.

VI. Two-Course Plain Concrete Floor

A. CONCRETE BASE

Proportions

48. The concrete shall be mixed in the proportions by volume of one (1) sack of portland cement, two and one-half (2½) cu. ft. of fine aggregate and five (5) cu. ft. of coarse aggregate.

Consistency

49. The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under slight tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

Placing

50. After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections of the required depth and width. Under no circumstances shall concrete that has partly hardened be used. The forms shall be filled and the concrete struck off and tamped to a surface the thickness of the wearing course below the established elevation of the floor. The method of placing the various sections shall be such as to produce a straight, clean-cut joint between them so as to make each section an independent unit. If dirt, sand or dust collects on the base it shall be removed before the wearing course is applied. Workmen shall not be permitted to walk on the freshly laid concrete. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used. In no case shall concrete be deposited upon a frozen subgrade or subbase.

Reinforcement

51. Slabs having an area of more than one hundred (100) sq. ft. shall be reinforced with wire fabric, or with plain or deformed bars. The reinforcement shall have a weight of not less than twenty-eight (28) lb. per one hundred (100) sq. ft. The reinforcement shall be placed upon and slightly pressed in the concrete base immediately after the base is placed. It shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal.

LIMITING CONDITIONS

Size of Slabs. Concrete, like most materials, expands and contracts with changes in temperature. Contraction of a floor laid on the ground induces tensile stresses because of the frictional resistance between the floor and the subbase. Where the floor is not provided with reinforcing, it has comparatively little tensile strength and is liable to crack if the slabs are too great in any dimension. Experience has indicated that 10 feet is the maximum size of floor slab that should be used where exposed to wide temperature changes unless reinforced to resist temperature stresses.

Protection of Edges. In certain conditions where concrete floors are subject to excessive abrasion the edges of slabs are liable to be badly chipped unless protected. For this purpose metal strips or angles are sometimes embedded in the edges of adjacent slabs or the edges rounded off and the crevice filled with a stiff joint filler.

TWO-COURSE PLAIN CONCRETE FLOOR

CONCRETE BASE

Reinforcement. As stated above plain floor slabs of the usual thickness and mixtures should not have dimensions greater than 10 feet between joints in order to insure against cracking caused by the tensile stresses developed by contraction. When it is necessary to place slabs with dimensions greater than 10 feet reinforcement should be provided to resist these tensile stresses. It is apparent that if the reinforcement is allowed to extend through an expansion joint, the purpose of the joint would be defeated, because the metal would prevent any movement at the joint. If reinforcement is not lapped sufficiently at its edges or joints, such joints would be planes of weakness and would allow cracks to form there.

B. WEARING COURSE

Proportions for Mixture No. 1 — 52. The wearing course shall be mixed in the proportions of one (1) sack of portland cement, two (2) cu. ft. of fine aggregate. The minimum thickness shall be three-quarters (¾) inch.

Proportions for Mixture No. 2 — 53. The wearing course shall be mixed in the proportions of one (1) sack of portland cement and one (1) cu. ft. of fine aggregate and one (1) cu. ft. of No. 1 aggregate for wearing course. The minimum thickness shall be three-quarters (¾) inch.

Consistency — 54. The mortar shall be of the dryest consistency possible to work with a sawing motion of the strikeboard.

Placing — 55. The wearing course shall be placed immediately after mixing. It shall be deposited on the fresh concrete of the base before the latter has appreciably hardened, and brought to the established grade with a strikeboard. In no case shall more than forty-five (45) minutes elapse between the time the concrete for the base is mixed and the wearing course is placed.

Finishing — 56. After the wearing course has been brought to the established grade by means of a strikeboard, it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. When required, the surface shall be steel-troweled, but excessive working shall be avoided. A mixture of dry cement, sand and number one aggregate may be applied to the fresh concrete of the base for a wearing course, but in no case shall dry cement or a mixture of dry cement and sand be sprinkled on the surface of the wearing course to absorb moisture or to hasten the hardening. Special methods not conflicting with these specifications may be used.

Coloring — 57. If artificial coloring is used, it must be incorporated with the entire wearing course, and shall be mixed dry with the cement and aggregate until the mixture is of a uniform color. In no case shall the amount of coloring exceed five (5) per cent of the weight of the cement.

VII. ONE-COURSE FLOOR

Proportions — 58. The concrete shall be mixed in the proportions of one (1) sack of portland cement to not more than two (2) cu. ft. of fine aggregate and not more than three (3) cu. ft. of coarse aggregate, and in no case shall the volume of the fine aggregate be less than one-half (½) the volume of the coarse aggregate.

A cubic yard of concrete in place shall contain not less than six and eight-tenths (6.8) cu. ft. of cement.

Consistency — 59. The materials shall be mixed with sufficient water to produce a concrete which will hold its shape when struck off with a strikeboard. The consistency shall not be such as to cause a separation of the mortar from the course aggregate in handling.

WEARING COURSE

Many attractive surfaces can be obtained in concrete floors by using colored aggregates or mineral coloring materials with either grey or white portland cement. An infinite variety of patterns can be set into the wearing course by using two or more different mortar mixtures. The Terrazzo or Venetian finish is often used in lobbies, corridors and show rooms. Marble or other small chips are added to the mixture for the wearing course and after it is laid additional chips are strewn upon the surface and rolled in. After the surface has hardened somewhat, it is ground down, thus obtaining a hard, smooth, polished surface consisting mainly of chips, very little of the cement mortar being visible.

A concrete floor can be prepared for dancing by waxing in a manner similar to that used for a wooden floor, or by several applications of liquid soap well rubbed into the floor, or by a special process that drives liquid wax into the concrete. Concrete floors can be protected from the action of acid solutions by several different treatments, the choice depending on local conditions.

One-Course Floor. Although usually a one-course floor cannot be given quite as smooth a surface as a two-course floor or any of the special finishes or treatments, yet where such finishes are not desired, it is considered the better type of floor, because being a monolithic slab, it is stronger structurally, and it can be laid more easily and quickly. In order to obtain a reasonably smooth surface a somewhat richer mixture should be used and more care should be taken with the finishing. If these precautions are observed a satisfactory and durable surface will be secured.

Placing

60. After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections to the required depth and width. Under no circumstances shall concrete that has partly hardened be used. The forms shall be filled and the concrete brought to the established grade with a strikeboard. The method of placing the various sections shall be such as to produce a straight, clean-cut joint between them so as to make each section an independent unit. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used. Workmen shall not be permitted to walk on the freshly laid concrete. In no case shall concrete be deposited upon a frozen subgrade or subbase.

Reinforcement

61. Slabs having an area of more than one hundred (100) sq. ft., or having any dimensions greater than ten (10) ft., shall be reinforced with wire fabric or with plain or deformed bars. The reinforcement shall have a weight of not less than twenty-eight (28) lb. per one hundred (100) sq. ft. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed. It shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal.

Finishing

62. After the concrete has been brought to the established grade by means of a strikeboard, and has hardened somewhat, but is still workable, it shall be floated with a wood float in a manner which will thoroughly compact it and provide an even surface. When required, the surface shall be steel troweled, but excessive working shall be avoided. Unless protected by metal, the surface edges of all slabs shall be rounded one-half (½) inch.

AMERICAN CONCRETE INSTITUTE STANDARD.

STANDARD SPECIFICATIONS FOR MONOLITHIC CONCRETE SEWERS AND RECOMMENDED RULES FOR CONCRETE SEWER DESIGN.*

Submitted by Committee S-3, on Reinforced and Plain Concrete Sewers and Conduits. (Revision of old Standard No. 24.)

PART I.—MATERIALS.

Cement.

Section 1. All cement shall conform to the current specifications for portland cement of the American Society for Testing Materials, and shall be tested in accordance with the methods of testing described in the specifications of that Society.

Fine Aggregate.

Section 2. Fine aggregate shall consist of sand graded from fine to coarse and passing when dry a screen having holes one-quarter in. in diameter. It shall be clean, coarse, free from dirt, vegetable loam or other deleterious matter. Not more than 6 per cent shall pass a sieve having one hundred meshes per lin. in.†

Section 3. Fine aggregate shall be of such quality that mortars composed of the proportions of cement and fine aggregate hereinafter specified for the various classes of concrete shall show a compressive strength after fourteen (14) days at least equal to the strength of mortar made of portland cement and standard Ottawa sand in corresponding proportions and of the same consistency‡.

Coarse Aggregate.

Section 4. The coarse aggregate shall consist of crushed stone, gravel or blast furnace slag which is retained on a screen having ¼ in. diameter holes and graded from the smallest to the largest particles. It shall be clean, hard, durable, free from all deleterious matter and soft, flat or elongated particles. Crusher dust in sufficient quantity to weaken the concrete will not be permitted. For reinforced-concrete arches or for plain

---

* Accepted by Annual Meeting, January, 1923. Adopted as Standard, by letter ballot, April 29, 1924.

†Crushed stone screenings may be permitted for use as fine aggregate provided that they shall comply with all the specifications of Sections 2 and 3, and further that they shall be produced from stone having a French coefficient of wear of not less than 8, as described in Bulletins No. 347 and 370 of United States Department of Agriculture.

‡It is recommended that, if possible, available fine aggregates be tested before awarding contracts. If it appears necessary to use an aggregate of poorer quality than above specified the proportion of cement in the various classes of concrete should be increased in order that the strength of the mortar actually used shall not be less than that with the Ottawa sand. If, however, the strength of the resulting mortars is less than 70 per cent of those with Ottawa sand, fine aggregate should be rejected entirely.

concrete arches less than 6 in. in thickness, the maximum size of particles shall be such as will pass a screen having 1 in. diameter holes. For inverts and plain concrete arches over 6 in. in thickness, the maximum size of particles shall be such as will pass a screen having 1½ in. diameter holes.

Section 5. Where crushed stone is used it shall have a French coefficient of wear of not less than 8, as described in bulletins No. 347 and 370 of United States Department of Agriculture.

(The above paragraph is for use in locations where limestone or sandstone of a questionable value are common. If all available stone it suitable, the paragraph may be omitted.)

Aggregate

Section 6. Samples of not less than ½ cu. ft. of fine aggregate and not less than one cu. ft. of coarse aggregate shall be delivered in suitable boxes or containers. All samples shall be plainly labeled with the places where taken, where to be used, the date, and the name of the collector.

Section 7. For the purpose of determining proportions of materials for concrete, each bag of cement containing 94 lb. shall be considered as containing one cu. ft. Sand and coarse aggregate shall be measured loose in approved boxes or hoppers.

Water.

Section 8. Water used for concrete shall be free from oil, acid, alkalies, or organic matter.

Concrete Reinforcement Bars.

Section 9. All steel reinforcement shall consist of cold drawn steel wire fabric having an elastic limit of not less than 55,000 lb. per sq. in.; or of expanded metal having an elastic limit of not less than 55,000 lb. per sq. in., and expanded cold from steel sheets; or of reinforcing bars.

9.1. Steel bars for reinforced-concrete sewers shall conform to the current specifications of the American Society for Testing Materials for (A) Billet Steel or (B) Rail Steel, except that rail steel bars may be used in sizes of 1 in. and under only, and hot twisted bars will not be permitted.

Section 10. Dimensions of bars given on the plans are based on square sections. The net area and weights of bars shall not be less than 95 per cent of the values for square bars as indicated. In computing the weights of steel, one cu. in. of steel shall be regarded as 0.283 lb.

Measurement and Payment

Section 11. The quantity of metal to be paid for shall be the number of pounds actually placed, as shown on the drawings as ordered. It shall not include any waste metal due either to the nature of the construction or to the fact that the lengths supplied are too long or too short for their purpose.

The quantity paid for shall, however, include extra metal in laps, where authorized, due to the fact that a single bar would be unreasonably long.

All bars shall be of the length ordered and shall be in one piece where required up to 30 ft. in length.

The compensations shall cover the cost of furnishing and delivering metal, including any royalty, the cutting, bending, placing, fastening in position, coating with cement and all other work and materials connected therewith.

Castings.

Section 12. Circular cast iron frames and covers for manholes and catch basins and any other iron castings shown on the drawings, or specified herein, necessary to complete the work, shall be furnished and placed. **Description**

Section 13. All castings shall be of tough, close-grained gray iron, free from blow-holes, shrinkage, and cold-shuts. They shall be sound, smooth, clean, and free from blisters and all defects. **Cast Iron**

Section 14. All castings shall be made accurately to dimensions to be furnished and shall be planed where marked or where otherwise necessary to secure perfectly flat and true surfaces. Allowances shall be made in the patterns so that the thickness shall not be reduced. Manhole covers shall be true and shall seat at all points. **Workmanship**

Section 15. All castings shall be thoroughly cleaned and painted before rusting begins, and before leaving the shop, with two coats of high-grade asphaltum or other suitable varnish that the engineer may direct. After the castings have been placed in a satisfactory manner, all foreign adhering substances shall be removed and the castings given two additional coats of asphaltum or other varnish as directed by the engineer. **Cleaning and Painting**

Section 16. No casting shall be accepted the weight of which shall be less than that computed to its dimensions by more than five per cent.

Material for Lining Inverts.

Section 17. All vitrified brick shall be uniform in size, and be not less than 8 in. by 4 in. by 2 in., nor more than 10 in. by 4½ in. by 2½ in. in length, width or thickness, respectively. The brick shall be free from lime or other impurities, uniformly vitrified and annealed, and shall have one edge face such that if the brick is laid on a horizontal plane on that face no portion thereof shall be more than ⅛ in. from the plane.

Section 18. Concrete block for sewer lining shall be uniform in size, not more than 18 in. by 12 in. in surface area and not less than 3 in. in thickness. They shall be made of Class "A" or better concrete, as hereinafter specified, in satisfactory molds, and thoroughly cured. They shall have an ultimate compressive strength at 28 days of not less than 2000 lb. per sq. in.

Section 19. Tile liners for inverts shall not be more than 8 in. by 12 in. in surface area, and not less than 2 in. in thickness. The back of the

tile shall be roughened and equipped with lugs or projections for bedding in mortar. They shall be manufactured under the general requirements covering vitrified sewer pipe and shall comply with the standard tests of the American Society for Testing Materials for clay sewer pipe in so far as applicable.

PART II.—CONCRETE FOR MONOLITHIC CONCRETE SEWERS.

Section 20. Concrete shall consist of a mixture of cement, fine and coarse aggregate and water of the qualities hereinbefore specified.

Concrete shall be of three classes, proportioned as follows:

Class	Cement	Fine Aggregate, cu. ft.	Coarse Aggregate, cu. ft.
A............................	1 sack	2	4
B............................	1 "	2½	5
C............................	1 "	3	6

Mixing

The relative proportions of fine and coarse aggregates may be modified at the direction of the engineer, provided that the proportions of cement to the total of the aggregates measured separately shall not be changed.

Section 21. Concrete shall be machine mixed. The concrete mixer shall be designed to take one completed batch of materials (using whole bags of cement) and to mix that batch thoroughly before any portion of it is withdrawn or any portion of the succeeding batch is introduced. The mixer shall be equipped with a tank so designed that when once set it will automatically supply to the mixer the amount of water so determined. The mixer shall be equipped with an instrument for measuring the time of mix.

Section 22. Concrete shall be mixed at least one minute after all the ingredients, including water, have been discharged into the mixer. Where the character of the work will permit, concrete shall be mixed in batches of one-half to one cu. yd. and the mixer speed shall not be less than 12 nor more than 19 revolutions per minute. Where small mixers are used, the speed shall not exceed 22 revolutions per minute.

Section 23. No concrete shall be hand-mixed except relatively small quantities, and then only by special permission of the engineer.

Section 24. Where concrete is mixed by hand, the cement and fine aggregate shall be mixed dry on a properly constructed wooden or steel platform built for the purpose until it shall have obtained an even and uniform color throughout. The mixture shall then be spread to make a bed of uniform thickness, on which shall be spread the coarse aggregate and the whole wet with the required amount of water and turned with square-pointed shovels at least three times or until a uniform mixture is secured, water being added from time to time, if necessary.

Section 25. In all plain concrete, where the thickness is 15 in. or more, there may be embedded broken pieces of sound stone, the greatest dimension of which does not exceed 6 in., and the least dimension of which is not less than three-quarters of the greatest dimension. These stones shall be set in the concrete as layers are being rammed, in a satisfactory manner, and so placed that each stone is completely and perfectly embedded. In general, there shall be a space of 4 in. between the stones and no stone shall come within 4 in. of any exposed face. The stone shall be thoroughly cleaned and wet before placing. **Rubble or Stone in Concrete**

Section 26. In mixing concrete, it is advisable to use the least possible amount of water required to obtain a workable mix, and when the aggregate is dry, 6 gal. of water to a sack of portland cement is the maximum which should be used. (For slag aggregate this may be somewhat increased.) Where comparatively dry mix is to be used, as in inverts, and near the crown of the arches, the concrete must be thoroughly tamped until the water flushes to the surface. **Consistency**

Section 27. Concrete shall not be mixed nor deposited in the work in freezing weather except as directed. If the work on concrete structures is prosecuted in cold weather, proper precautions shall be taken for removing ice and frost from the materials, including heating the water and aggregates; for protecting the newly-laid masonry from freezing, and for securing work satisfactory in all respects. Satisfactory covering for the newly-laid concrete and such additional appliances and materials as may be required therefor, including steam pipes for keeping the air warm beneath the said covering shall be provided. **Work in Freezing Weather**

Transporting and Placing Concrete.

Section 28. Provision shall be made for quickly transporting the concrete from the mixer to the work and with as little shaking as possible, so that the tendency of water to rise to the top may be reduced to a minimum. Any concrete which may have been compacted during transportation shall be satisfactorily remixed before being placed in the work. Any concrete delayed one-half an hour in transit shall not be used in the work and must be removed from the premises. **Transporting**

Section 29. Concrete shall be deposited so as to maintain a nearly level surface and avoid flowing along the forms. It shall be continuously and sufficiently worked to expel air and to force the aggregate away from the forms. In special cases, as where concrete is deposited on slopes, a comparatively dry mixture may be used, but great care shall be exercised to spread such concrete evenly in layers not more than 6 in. in thickness and to ram it thoroughly. In general, the methods used shall be such as to give a compact, dense and impervious concrete with a smooth surface. **Placing Concrete**

Joining New Work to Old

Section 30. For the proper bonding of new and old concrete, such provisions shall be made of steps, dovetails, or other devices as may be required. Whenever new concrete is joined to old, the contact surface of the old concrete shall be thoroughly cleaned, using a stiff brush and a stream of water, if required, and shall be clean and wet at the moment the fresh concrete is placed. Where ordered, a thick wash of rich mortar shall be run over the contact surface of the old concrete. Where it is of importance that the joint between the new and old work shall be as strong and tight as possible, especial precautions shall be taken, such as picking off the top one or two in. of the old work so as to remove the laitance or washing the old cement off the surface with acid or alkali and later with water to remove all traces of them, or both, as may be required.

Finish of Concrete Surfaces

Section 31. Special care shall be taken that all concrete surfaces shall be smooth and free from indentations or projections. All surfaces shall be free from voids, exposed stones and other imperfections. If such imperfections are found upon removing the forms, the faults shall be corrected at the contractor's expense by filling with mortar or otherwise, as directed, even to the extent of taking down and replacing unsatisfactory concrete.

Plastering of Concrete Surfaces

Section 32. No plastering of any concrete surface shall be done unless expressly permitted and if so permitted shall be done in strict accordance with directions. No payment will be made for plastering done to correct defective work.

Masonry not to be Laid in Water

Section 33. No concrete or other masonry shall be deposited under water without permission and then only in accordance with directions. Water shall not be permitted to rise on any masonry until the mortar shall have set at least 12 hours.

Forms.

Forms

Section 34. There shall be provided suitable collapsible centers or forms with smooth surfaces of ample strength and rigidly braced. The bracing shall be adequate to prevent deviations from the correct lines.

Section 35. All steel forms shall be neatly and accurately made with all similar parts in each longitudinal section of form interchangeable with other sections. Bent plates required to fit shall be rolled and fabricated to the correct curves before assembling. Suitable forms shall be provided for bends in the sewer. Steel filler plates shall be furnished.

Section 36. All wooden forms shall be built of clean, sound lumber, reasonably free from knots, dressed on all sides of uniform thickness and neatly fitted. Tongued and grooved material shall be used where required. The form surface shall be watertight, securely fastened to the ribs or supports.

Section 37. No forms built up in the trench or ribs with separate pieces of wooden lagging, piece by piece, will be allowed except for specials or curves.

Section 38. No center or form shall be used which is not clean and of proper shape and strength and in every way suitable. Before placing concrete or reinforcement the forms shall be coated with vaseline, form grease or other suitable approved substance, to prevent adherence of concrete.

Placing Reinforcement.

Section 39. All steel reinforcement shall be placed in the exact positions and with the spacing shown on the drawings or as ordered, and it shall be so fastened in position as to prevent displacement while the concrete is being deposited. **Placing Concrete**

Section 40. The reinforcing steel shall be bent to the shapes shown on the drawings or as required. The ends of the bars shall be bent or hooked over if required. The length of the laps for bonding the adjacent bars shall not be less than thirty times the diameter of the bar, when the steel is designed for working stress of 12,000 lb. and not less than forty times the diameter of the bar when the steel is designed for working stress of 16,000 lb. per sq. in. Where the bars are of different sizes, the diameter of the larger bar shall be used. **Shaping and Splicing**

Section 41. Steel must be stored in such manner that its condition will at all times correspond to that under which the samples were taken. **Storing**

PART II-A.--GENERAL CONSTRUCTION, MONOLITHIC SEWERS.

Section 42. Below the springing line for such sewer, the trench shall be accurately shaped to the form of the outside of the masonry and the concrete shall have a firm bearing on the natural soil or rock at all points below the springing line.* **Width of Trench**

Section 43. In general the width of the trench for sewers of the horseshoe and similar types shall be one ft. greater than the outside width of masonry to allow for satisfactory bracing.

Section 44. Underdrains of agricultural tile laid in gravel or crushed stone, shall be constructed of the size, and where directed, for the purpose of keeping the work free from water during construction, such drains to be abandoned when the work is completed; underdrains so laid shall lead to sumps or manholes, and water flowing to them shall be removed by pumping. Such pumping shall be carried on continuously, day and night, and the level of the ground water shall be maintained below any cement or concrete which may be placed in the work for a period of at least twelve hours after such cement or concrete is placed. When the temporary underdrains above described are abandoned, they shall be cut and plugged where directed and the sump holes above described shall be solidly filled with approved material. **Underdrain**

Construction of Inverts

Section 45. On all sections having a comparatively flat invert, the complete invert shall first be built, while on all circular sections a center strip of not less than one-fourth circumference shall be built. The invert or center strip shall be placed in sections of not over 16 ft. where the surface is to be finished with end guides and a longitudinal straight edge, and not more than twenty ft. if a separate lining of vitrified brick, tile or concrete block is to be provided.

Finish

Section 46. A granolithic finish shall be applied to the fresh concrete as soon as the condition of its surface will permit. This finish shall consist of a mixture of one part of cement to two parts of granite, or other hard rock chips, graded from 1/8 in. to 1/2 in. in size, and shall be laid 1 1/2 in. thick. The upper surface shall be formed by means of screeds and shall be floated and troweled to a smooth surface. As soon as this surface is dry enough to receive it, a dry mixture of two parts of cement and one part of sand, free from crusher dust and particles larger than 1/8 in. shall be sprinkled over it and then the surface shall be floated and troweled. This treatment shall be repeated at least once, and where the proportion of very fine material in the aggregate necessitates it, a total of three dryer coats shall be applied. Where the placing of the dryer coat must be deferred until the day following the pouring of the concrete invert, the concrete shall be first moistened and covered with neat cement, which shall be thoroughly broomed into the concrete in the form of a thick paste.

Lining (Alternate to Sec. 46)

*Section 47.** Where required, the inverts shall be lined with concrete block, tile, or vertical brick, as shown on the plans.

Block Lining

47.1. The concrete bottom shall be accurately shaped up to a line one-half in. below the bottom of the lining and allowed to set before the lining is laid.

A mixture made of one part of portland cement and three parts of sand, without the addition of water, shall be spread on the finished surface to the depth required to bring the block or brick to the required grade. The lining units shall be laid in straight lines and in a workmanlike manner and so that all joints shall be broken. After being laid, it shall be rolled with a hand-roller weighing from 300 to 500 lb. or tamped until every unit shall have a solid bearing and the top of the finished work shall present a smooth and even surface and conform accurately with the shape of the invert as shown on the plans. The joints between the units shall be grouted with mortar made of one part portland cement and two parts sand and the surface shall be brushed until every joint is completely filled.

Tile Lining

47.2. Where vitrified tile is used, the units shall be carefully bedded in wet mortar, the mortar bed to be approximately 1/2 in. in thickness.

Protection

47.3. The bottom must be kept free from water until the work is completed and no water will be allowed to run over the completed work until it shall have set.

---

*Where there is a probability that wet ground will make the shaping of circular inverts impossible, an alternate section of a suitable type shall be used.

Section 48. The unfinished surface of the invert on which the concrete of sidewalls or arches is to be placed, shall be made as rough as possible. In unreinforced work, dovetails shall be formed as provided in Section 31. In reinforced work, where projecting bars may interfere with the formation of dovetail joints, the invert concrete shall be thoroughly cleaned by a pressure stream of water or scrubbed and every precaution shall be used to prevent earth or material from the forms falling on the surface after cleaning.

Keeping Concrete Moist

Section 49. Precaution shall be taken to prevent concrete from drying until there is no danger of cracking from lack of moisture. Concrete shall be kept moist for at least one week, unless sooner covered with earth. This may be done by covering of wet sand, burlap, continuous sprinkling or by some other method approved by the engineer.

Section 50. Forms for slabs or very flat arches as in box sections or roofs of special chambers, shall remain in place for at least seven days. No load shall be placed on the concrete for fourteen days, and then only with permission.

Removal of Arch Forms

Arch forms shall not be slackened until the backfilling has been carried to a height of at least one ft. above the top of the arch and tamped. Arch forms shall remain in place for forty-eight hours when conditions are most favorable for the hardening of the concrete and for a longer time, as may be directed during inclement weather, or where unusual conditions exist. Permission for dropping center must be secured for each arch unit.†

Section 51. Backfill, over and around arch sewers, shall be placed as soon as possible after the cement has set. The filling up to a plane 2 ft. above the top of the arch shall be made from the best earth and shall not contain a sufficient amount of large stones as to allow the pieces of stone to become wedged. It should be filled in layers of not over 6 in. and carefully tamped. If the remaining of the backfill is dumped from buckets, the contents of the buckets should not be allowed to fall more than 5 ft. unless the impact is broken by timber grillage. Bracing should generally be removed only when the trench below it has become completely filled and every precaution shall be taken to prevent any large slips of earth from the side of the trench onto or against the green arch. All voids left by withdrawal of sheeting shall be immediately filled with sand, by ramming with tools especially adapted to that purpose, by watering or otherwise as may be directed.

Section 52. During the construction of the sewer, care should be taken that no loose mortar or concrete shall be allowed to remain on the interior surface of the invert. At the completion of the work all débris shall be removed and the invert shall be left clean and smooth.

---

*This construction is particularly applicable to sewers having comparatively flat inverts, and is more difficult to carry out with circular sewers.

†For small arches, 6 ft. or less, and under the most favorable conditions, forms may be dropped in 24 hours.

General Note.

No attempt has been made to include herein detailed specifications, for much of the work entered into sewer construction, such as earth and rock excavation, sheeting and bracing, etc.

Statements relating to the responsibility of furnishing materials, performing work and giving directions have also been generally omitted.

No specification has been included in regard to the measurement of materials or the amount of work covered in any particular compensation, other than for reinforcing steel, as it has been considered best to leave these clauses to be worked under the conditions prevailing on the particular piece of work.

RECOMMENDED RULES FOR CONCRETE SEWER DESIGN.

(To accompany the Specifications for Concrete Sewers.)

Plain Concrete Monolithic Sewers

1. Concrete sewers without reinforcement are approved for sizes between 30 and 60 in. mean diameter. Plain concrete sewers between these sizes are to be used only in rock or hard soils. It is recommended that the minimum thickness for a diameter of 36 in. or under should be 5 in. and for a 5 ft. diameter 7 in. with intermediate sizes in proportion. These thicknesses are to be taken as a minimum for circular sewers and used only under favorable conditions.

Reinforced Concrete Monolithic Sewers

2. All sewers near the surface and subject to moving loads or vibration, should be reinforced. For sewers of 6 ft. or less in diameter, it is recommended that the reinforcement be ½ of 1 per cent placed near the inside at the crown, and near the outside at the springing lines.

If it appears at all possible that the horizontal pressures on the sewer might be large, reinforce for reverse stresses.

3. It is recommended that for all sewers greater than 6 ft. in diameter, several possible types of loading be assumed and stresses be calculated on the elastic arch theory. (The methods are indicated in Turneaure & Maurer's "Principles of Reinforced Concrete," or in Metcalf & Eddy's "American Sewerage Practice," Volume I.)

4. It is also suggested that in sewers of greater than 6 ft. in diameter, it may be found economical to adopt a section having a comparatively flat bottom, and an arch with or without intermediate side walls.

5. The minimum thickness of concrete in sections of this type should be 8 in. This is recommended as a factor of safety against poor placing and also to secure warterproof structures.

6. The specifications submitted provide for three classes of concrete. It is recommended that all arches be built of Class "A" concrete and that the inverts be of Class "A" concrete except in rock or very hard soils, where Class "B" concrete may be used.

7. For reinforced work in bad ground, the designer should provide for a raft of Class "C" concrete of from 4 to 6 in. in depth, which is to be allowed to set before the reinforced structure is started. This is advisable

to facilitate good workmanship and particularly to prevent contamination of the concrete around the reinforcement by mud or sand.

8. The distance from the face of reinforcing steel to the face of the concrete in monolithic sewers should be not less than 2 in.

9. In determining dimensions of concrete and reinforcement, the following working stresses should be the maximum used:

(*a*) The maximum workin stress in the steel where structural grade is used should be not more than 12,000 lb. and for intermediate or hard grades, or for cold twisted bars, 16,000 lb. per sq. in.

The maximum working stress in rail steel should not exceed 16,000 lb. per sq. in.

(*b*) The maximum working stresses in concrete are based on the Report of the Joint Committee on Concrete and Reinforced Concrete are about 25 per cent less than the stresses there recommended.

WORKING STRESSES IN POUNDS PER SQUARE INCH.

Aggregate	Class A	Class B	Class C
Granite or trap rock	550	450	350
Gravel or hard limestone	500	400	325
Soft limestone or sandstone (if permitted)	375	300	250

Class "B" concrete is not recommended for use in the sewer proper. Soft limestone and sandstone are prohibited if the accompanying specification is rigidly carried out.

These stresses should be further reduced where construction conditions are likely to be very unfavorable to good workmanship, as in very wet or deep trenches.

10. In all important work, specify that the reinforcement shall be held in place with steel chairs or holders and wire ties.

11. Attention is called to the fact that with sewers having comparatively flat inverts, careful consideration must be given to the distribution of load across the invert. Where soils are likely to be compressible, the weight should be taken as uniformly distributed. The stresses in such inverts should be carefully analyzed, as they are generally more severe than the other parts of the sewer.

It will generally be advisable to provide alternate details of the invert for use in rock cuts, when resting on rock or nearly incompressible soils and for soft or wet ground.

12. Accompanying specifications for monolithic work provide for either a granolithic finish on the invert or for a lining of concrete block, brick or tile. The use of the separate lining should be considered as an additional factor of safety where unsatisfactory construction conditions are likely to affect adversely the quality of workmanship and the strength or density of the finished invert concrete.

AMERICAN CONCRETE INSTITUTE

BUSINESS REPORTS

ANNUAL REPORT OF THE BOARD OF DIRECTION TO THE MEMBERS OF THE AMERICAN CONCRETE INSTITUTE, TO JANUARY 1, 1923.

REPORT OF BOARD OF DIRECTION.

The Institute has continued in the last year to show progress in membership, financial condition and in technical activity. A summary of gains shows.

Feb. 1, 1920	Membership	428
Feb. 1, 1921	Membership	627
Feb. 1, 1922	Membership	806
Feb. 1, 1923	Membership	981
Feb. 1, 1924	Membership	1161
June 1, 1924	Membership	1272

The interests of these members are highly diversified within the very broad field of concrete. It is gratifying that students of concrete in practically every department of interest are giving increasing recognition to the Institute's possibilities for service to them.

Members of the Institute have been good advertisers of the Institute's work and the secretary's office has had very generous co-operation in efforts to increase the membership of the organization. The work of the 20th Anniversary Committee, under the Chairmanship of Mr. H. C. Turner, and with the very active working direction of Mr. J. P. H. Perry, a vice-chairman, has been a very important factor in bringing the Institute to the favorable notice of a considerable number who have not hitherto been actively with us. The active work of a special Chicago membership committee, headed by Mr. George Warner, has brought us many new and valuable friends.

The Institute's work is gathering momentum. It is getting closer to the problems of concrete as they are met with in the field. This makes its membership advantages more obvious to the busy man who first of all is interested in the progress of his particular work.

The finances of the Institute are improving. Our most formidable item of expense is the publication of our Proceedings. For some years their cost each year has been met out of the income of the succeeding fiscal year. We are now up to date. While our Volume 20, to be published as soon as possible after this convention, will doubtless be unusually expensive because of the large bulk of valuable papers and reports, our finances are in such a condition that we expect to be able to meet the expense this year and still show a small surplus, without touching the income of the next fiscal year.

It has been the policy of the Institute to base its annual budget on a figure very close to anticipated income with a small reserve of about $3000

in Government securities against a possible "rainy day." It has been the thought of the Board that it would be the wish of the members that while conducting our affairs conservatively, the Institute should spend its income to strengthen the Institute's position as an educational organization, giving the greatest possible service to the greatest number.

The report of our auditor for the fiscal year ended June 30, 1923, is as follows:

ALBERT E. HORNE
PUBLIC ACCOUNTANT
DETROIT

DETROIT, MICH., August 10, 1923.

Mr. Harvey Whipple, Treas.,
American Concrete Institute,
East Grand Blvd. and Moran,
Detroit, Michigan.
DEAR SIR:

In accordance with your instructions, we have made an examination of the books and records of the American Concrete Institute, for the period from June 30, 1922, to June 30, 1923, for the purpose of verifying the cash transactions of the period and presenting the financial condition of the Institute at that date.

We submit herewith two exhibits as follows: Exhibit "A" Statement of condition as of June 30, 1923. Exhibit "B" Statement of receipts and disbursements from June 30, 1922 to June 30, 1923.

The cash in bank amounting to $4513.46 as shown by the Cash Book was verified by reconciliation with the Statement rendered by your depository as of June 30, 1923. During our examination paid checks all of which were properly approved were seen for all disbursements.

The remaining items in statement of condition are shown in accordance with the records and have not been further verified by us.

We have not attempted to determine the income which should have been derived during the year and have restricted our examination to the disposition of all cash shown to have been received.

We hereby certify:

1. That all cash shown to have been received has been accounted for and that we have seen satisfactory evidence of payment for all disbursements.

2. That the cash in bank, amounting to $4513.46, June 30, 1923, was on deposit on that date and the Imprest Cash amounting to $300.00 was in the hands of the Treasurer.

3. That the Statement of condition (Exhibit A) is in accordance with the records and subject to the foregoing comments, in our opinion properly presents the financial condition of the American Concrete Institute on June 30, 1923.

Respectfully submitted,

(Signed) A. E. HORNE.

EXHIBIT "A"

STATEMENT OF CONDITION.

June 30, 1923.

ASSETS.

Cash in Bank	$4,513.46	
Cash Imprest	300.00	
Total Cash		$4,813.46
U. S. Treas. Certificates		3,000.00
Accounts Receivable:		
Active Members	$1,380.00	
Contributing Members @ $50.00	1,075.00	
Special	1,000.00	
Miscellaneous	75.93	
		3530.93
Inventories:		
Proceedings Prior to 1919—437 @ $0.50	$219.50	
Proceedings, 1919-1922—518 @ $1.00	518.00	
Proceedings, 1923—300 @ $3.50	1,050.00	
Journals, 1914-15—491 @ $0.10	49.10	
		1,835.60
Total Inventories		$13,179.99

LIABILITIES.

Concrete Building Units Fire Test Fund	$272.51	
Accrued:		
Printing Proceedings, 1923	3,543.70	
Reserve:		
For Loss of Delinquent Members	1,200.00	
Dues Paid in Advance	70.00	
Surplus	8,093.78	
		$13,179.99

EXHIBIT "B."

CASH RECEIPTS AND DISBURSEMENTS.

July 1, 1923, to June 30, 1923.

As Shown by Cash Book.

RECEIPTS.

Cash on hand, June 30, 1922	$4,186.53
Dues Active	8,055.39
Dues Contributing	3,789.98
Dues Special	1,000.00
Proceedings	1,196.50
Concrete Building Units Fire Test Fund	60.00
Interest on Commercial Deposits	70.89
Interest on Victory Bonds	71.25
Misc. Preprints	717.71
Victory Bonds Redeemed	3,000.00
Total Assets	$22,148.25

DISBURSEMENTS.

Convention	$768.64	
Miscellaneous Preprints	1,245.97	
Office Expense	511.99	
Postage	741.49	
President's Office	11.18	
Printing, Stationery and News Letters	1,732.49	
Proceedings	4,332.36	
Rent	379.92	
Exchange	2.76	
Salaries	4,065.06	
Traveling Expense	328.00	
Misc. Expense	32.72	
National Fire Protection Association	60.00	
Auditor	37.50	
Bond for Secretary-Treasurer	50.00	
Concrete Building Units Fire Test	442.49	
U. S. Treasury Certificates	3,000.00	
	$17,742.57	
Less Disct. Recd.	107.78	
Total Disbursements		$17,634.79
Cash in Bank, June 30, 1923		$4,513.46
Cash Receipts, July 1, 1923, to January 31, 1924, have been		$13,127.20
Disbursements, July 1, 1923, to January 31, 1924:		
Charged to 1922-23 Budget	$3,829.28	
Charged to 1923-24 Budget	5,281.01	
Treasury Certificates	5,500.00	
		14,610.29
We had in bank, January 31, 1924		$3,030.37

With additional funds available, one problem is to relieve the tension in our annual convention programs. It is becoming each year a more serious problem to make the convention, in the sessions of four days, provide an outlet for everything of technical value our increasing membership and our increasingly active committees produce. The Board of Direction will ask the co-operation of committees in releasing progress reports through the year as rapidly as they can be made available—through our News Letters, thus promoting discussion and leaving for our convention the discussion of final reports and of those debatable matters which stand in the way of final reports in committee deliberations. In this we hope to have the benefit of early committee work, immediately following the convention so as to make our contact with members more nearly continuous throughout the year.

LIST OF REGISTRANTS.

An asterisk (*) designates a member.

*Abrams, Duff A., Lewis Institute, 1951 Madison St., W., Chicago, Ill.
*Acheson, William M., Div. Engr., New York State Highway Comm., Syracuse, N. Y.
*Affleck, B. F., Universal Portland Cement Co., Chicago, Ill.
*Ahlers, John G., Barney-Ahlers Constr. Corp., 110 W. 40th St., New York City.
*Aldridge, E. V., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
Allan, W. D. M., Portland Cement Association, Chicago, Ill.
*Allen, Cleveland, Francisco & Jacobus, 4 West 43rd St., New York City.
*Allen, Leslie H., 405 Broadway, Milwaukee, Wis.
Allison, Lyman J., 115 S. Dearborn St., Chicago, Ill.
Anderson, G. K., Official Stenographer, 5716 Cedar Ave., Philadelphia, Pa.
*Alpha Portland Cement Co., Easton, Pa., (Louis Anderson, Jr., Chem. Engr.)
Armiger, Boyd H., Bldg. Sup., McLennan Constr. Co., 237 Delaware Place, Chicago, Ill.
Arms, L. M., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
Arronet, Margaret G., Lewis Institute, 1951 W. Madison St., Chicago, Ill.
Ashelford, Herman, W. H. Ashelford & Sons, Byron, Ill.
Ashelford, Walter, W. H. Ashelford & Sons, Byron, Ill.
Ashelford, W. H., W. H. Ashelford & Sons, Esmond, Ill.
*Ashton, Ernest, Lehigh Portland Cement Co., Allentown, Pa.
Austin, R. W., Supt. Orr & Miller, Lima, Ohio.
*Babbitt, A. B., Gen. Mgr., The Kent Machine Co., Kent, Ohio.
Bailey, C. F., Anchor Con. Machinery Co., Milwaukee, Wis.
*Bailey, L. D., Elk River Concrete Products Co., Elk River, Minn.
*Baker, John C., Onondaga Litholite Co., 102 N. Beech St., Syracuse, N. Y.
Banks, Wm., (Cement products mfr.), Durant, Iowa.
Barker, Wm. E., Portland Cement Association, 1537 Conway Bldg., Chicago, Ill.
*Barriball, Geo. D., 13002 Miles Ave., Cleveland, Ohio.
*Bartlett, G. S., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
Bass, Verner, Chaldron, Nebr.
*Bell, O. P., Sunbury Cement Products Co., Sunbury, Pa.
*St. Paul Cement Works, 34 E. Forest St., St. Paul, Minn. H. C. Berchem.
*Berchem, H. C., St. Paul Cement Works, 34 E. Forest St., St. Paul, Minn.
Bergland, Irving, Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
*Bernier, N. M., N. Bernier & Son, 411 Walden St., Cambridge, Mass.
Berns, M. A., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
*Bertin, R. L., White Constr. Co., 95 Madison Ave., New York City.
Bille, Harold W., Atlas Portland Cement Co., Chicago, Ill.
*Binswanger, S. J., 5520 S. Park Ave., Chicago, Ill.
Black, J. C., Editor, "Engineering and Contracting," 221 E. 20th St., Chicago, Ill.
*Black, L. E., Solvay Process Co., Chicago, Ill.
Black, Roy, Builder, Cleveland, Ohio.
Blaine, Ethel, Asst. Secy., American Concrete Institute, Detroit, Mich.
*Blanchard, F. E., 75 E. Tompkins St., Columbus, Ohio.
Blisten, Sidney, Trade Press Pub. Corp., 413 15th St., Moline, Ill
*Block, H. L., The Cement Products Co., 911 E. High, Davenport, Iowa.
*Bloecher, Walter P., Stone & Webster, Inc., 142 Langdon Ave., Watertown, Mass.
Boardman, John W., Huron Portland Cement Co., Detroit, Mich.
*Bosch, Jacob, Calumet Concrete & Material Co., 100 W. 115th St., Chicago, Ill.

BOURNE, CHARLES LUTHER, Portland Cement Association, 111 W. Washington St., Chicago, Ill.
*BOWES, A. S., Editor, "Cement and Engineering News," 22 Quincy St., Chicago, Ill.
BOYAJOHN, H. M., Boyajohn & Barr, 503 Branson Bldg., Columbus, Ohio.
BOYD, GEORGE E., Editor, "Railway Review," 537 S. Dearborn St., Chicago, Ill.
BOYDEN, COL. H. C., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
*BOYER, E. D., Atlas Portland Cement Co., 25 Broadway, New York City.
BRADSHAW, GEORGE L., Independent Block & Cement Co., 2102 S. Harding St., Indianapolis, Ind.
*BRAUN, WALTER, 1083 S. High St., Columbus, Ohio.
BREWER, ARTHUR V., 219 W. 12th St., Flint, Mich.
BREWER, R. D., Portland Cement Assn., 111 W. Washington St., Chicago, Ill.
*BRICKETT, EDWARD M., 300 Lab. of Applied Mechanics, University of Illinois, Urbana, Ill.
BROWN, HERMAN E., Consumers Supply Co., Milwaukee, Wis.
BROWN, J. F., Illinois Steel Co., South Works, South Chicago, Ill.
*BROWN, R. P., National Lime Assn., 918 G St., N. W., Washington, D. C.
*BROWNING, ALBERT H., 3711 Osgood St., Chicago, Ill.
BUCKMAN, GEORGE H., Northwestern States Portland Cement Co., Charles City, Iowa.
*BUENTE, C. F., Concrete Products Co., of America, Pittsburgh, Pa.
BURGESON, MILDRED, Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
*BUTLER EQUIPMENT CO., Waukesha, Wis., Morgan R. Butler.
CALVIN, J. J., Truscon Steel Co., Youngstown, Ohio.
CAMPBELL, H. COLIN, Portland Cement Association, 111 W. Washington St., Chicago, Ill.
*CAMPBELL, S. A., Ridge Constr. Corp., 251 Elmdorf Ave., Rochester, N. Y.
CANNON, L. L., Cannery Equip. & Supply Co., Room 711, 211 Grand Ave., Milwaukee, Wis.
*CAMPION, H. T., McClellan & Junkersfeld, Inc., 61 Trinity Place, New York City.
CARLSON, JULIE V., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
CARR, THOMAS, Universal Portland Cement Co., Chicago, Ill.
*CAREY CONCRETE CO., Wisconsin Rapids, Wis., W. H. Carey, Pres.
*CARVEL, F. G., Calumet Steel Co., 208 S. La Salle St., Chicago, Ill.
*CATLETT, RICHARD H., Security Cement & Lime Co., Hagerstown, Md.
CERNY, JAMES, 1107 S. La Salle St., Chicago, Ill. (American System of Reinforcing.)
*CHAPMAN, CLOYD M., Dwight P. Robinson & Co., 171 Madison Ave., New York City.
CHILL, CHARLES W., The Detroit Edison Co., 2000 Second Ave., Detroit, Mich.
*CHITTENDON, HOWARD L., Clinton, Conn.
CHRISTIAN, W. A., 207 N. Parkside Ave., Chicago, Ill.
*CHRISTENSEN, EINAR, 210 E. Ave., Rochester, N. Y.
CHRISTIE, H. A., Gen. Supt., Raymond Concrete Pile Co., 923 Munsey Bldg., Baltimore, Md.
CHRISTOFFEL, JOHN, JR., Christoffel Concrete Products Co., 1023 18th St., Milwaukee, Wis.
*CHUBB, JOS. H., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
CLANCEY, IRENE, Portland Cement Association, Chicago, Ill.
*CLARK, H. P., Corrugated Bar Co., Buffalo, N. Y.
*CLARK, ROBERT J., Hinckley, Ill.
CLARKE, H. V., The Philip Carey Co., 3611 Loomis Place, Chicago, Ill.
CLARK, MILTON T., Truscon Steel Co., Youngstown, Ohio.
CLEVE, ALBERT, (Portland Cement Assn.), 3823 N. Laundale Ave., Chicago, Ill.
CLUGSTON, C. D., Sales Mgr., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.

COLGAN, C. H., Gen. Supt., Raymond Concrete Pile Co., 140 Cedar St., New York City.
*COLLIER, B. C., Cement Gun Co., Inc., Allentown, Pa.
*COLMAR, DANIEL, Ramloc Stone Co., Inc., Albany, N. Y.
CONAHEY, GEORGE, Field Manager, Portland Cement Assn., New York City.
*CONDRON, T. L., Condron Company, 53 W. Jackson Blvd., Chicago, Ill.
CONGER, WALTER C., Truscon Steel Co., Youngstown, Ohio.
*CONWAY, H. W., The Barriball Bros. Co., 4103 E. 100th St., Cleveland, Ohio.
*COOKE, C. E., Kalman Floor Co., Wrigley Bldg., Chicago, Ill.
*COOKE, GEORGE F., Kalman Floor Co., 11 E. 42nd St., New York City.
*COOPER, G. E., Barnesville, Ohio.
*COOPER, GILBERT, Ideal Concrete Constr. Co., Joliet, Ill.
CORNWALL, P. B., Alpha Portland Cement Co., 140 S. Dearborn St., Chicago, Ill.
*COX, G. L., Cement Products Co., P. O. Box 935, Birmingham, Ala.
*COX, LAWRENCE, Secy., Concrete Units Co., Hammond, Ind.
*CRABBS, AUSTIN, President, The Cement Products Co., Davenport, Iowa.
CRAIG, H. W., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
CREPPS, R. B., Prof. Testing Materials, Purdue University, W. Lafayette, Ind.
CROM, J. M., Cement Gun Co., Inc., 549 W. Wash., Chicago, Ill.
*CRONKRITE, A. C., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
*CROSS, HARDY, University of Illinois, Urbana, Ill.
*CRUM, R. W., Iowa State Highway Commission, Ames, Iowa.
*CUMMINGS, A. E., 452 Fullerton Parkway, Chicago, Ill. (Raymond Concrete Pile Co.)
*CURTIS, A. J. R., Portland Cement Association, Chicago, Ill.
*DAVIS, HERBERT A., Brooklyn Crozite Brick Corp., 120 W. 42nd St., New York City.
DEVOS, A. W., Wm. H. Devos Co., 3115 Ann Ave., Milwaukee, Wis.
*DEINBOLL, F. K., 1263 Brookley Ave., Cleveland, Ohio.
DIENHART, E. W., Portland Cement Association, Chicago, Ill.
*DONALDSON, W. E., Carnegie Steel Co., Pittsburgh, Pa.
DOUTHETT, C. L., Humboldt Gravel & Tile Co., Humboldt, Iowa.
*DOWDALL, E. J., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
*DOYLE, W. T., 128 N. Wells St., Chicago, Ill.
DUBREEL, E. F., Engineer, Cincinnati, Ohio.
*DUCKETT, N. S., 12 Castlefield Ave., Toronto, Ont.
*DUNN, J. A., Independent Concrete Pipe Co., 201 N. West St., Indianapolis, Ind.
DUNDAS, F. C., Portland Cement Association, Chicago, Ill.
*DUNN, H. E., W. E. Dunn Mfg. Co., Holland, Mich.
*DUNNELLS, C. G., Carnegie Institute of Technology, Pittsburgh, Pa.
*DURGIN, F. L., JR., Cramp & Co., 801 Denckla Bldg., Philadelphia, Pa.
*DWYER, J. R., U. S. Bureau of Standards, Washington, D. C.
EDWARD, G. G., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
*EARLEY, JOHN J., 2131 G St., N. W., Washington, D. C.
EGBERT, D. R., International Trade Press, Monadnock Blk., Chicago, Ill.
EGE, C. R., Mgr., Highways Bureau, Portland Cement Association, Chicago, Ill.
*EGELHOFF, R. F., 11 Goodell St., Buffalo, N. Y.
EKBLAW, K. J. T., Portland Cement Association, Chicago, Ill.
ELDRIDGE, BYRON H., 216 S. Ashland Ave., La Grange, Ill.
EMERSON, H. B., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
ENBLOM, ALBERT, Nelson Enblom Co., 917 Ply. Bldg., Minneapolis, Minn.
FROELIDE, WILHELMINA, Universal Portland Cement Co., Chicago, Ill.
*FAIRCHILD, L. J., Eastman Kodak Co., 3195 Lake Ave., Rochester, N. Y.
FARMER, H. G., Engineer, Universal Portland Cement Co., 526 Frick Bldg., Pittsburgh, Pa.
*FECHHEIMER, S. M., 615 Wayne St., Detroit, Mich.
*FELLOWS, A. W., Alpha Portland Cement Co., Easton, Pa.

*FERGUSON, JOHN A., 1012 Portland St., Pittsburgh, Pa.
FLATTER, C. P., Flatter Bros. Co., Green Bay, Wis.
FLODIN, H. L., Portland Cement Association, Chicago, Ill.
FORD, C. L., Chemist, Structural Materials Research Lab., 1951 W. Madison St., Chicago, Ill.
FOSHINBAUR, V. G., Portland Cement Association, Chicago, Ill.
*FOURNIE, ARTHUR J., Swansea Stone Works, R. R. 3, Belleville, Ill.
FOURNIE, CLEM, Swansea Stone Works, R. R. 3, Belleville, Ill.
FOURNIE, FRED, Swansea Stone Works, Belleville, Ill.
*FRANKLIN, JACK, 1218 Wrigley Bldg., Chicago, Ill.
FRANKS, C. D., Dist. Engr., Portland Cement Assn., 610 Merchants Bank, Indianapolis, Ind.
*FREEMAN, J. E., 122 S. Michigan Ave., Chicago, Ill.
*FREEMAN, P. J., 311 Ross St., Pittsburgh, Pa.
FREES, H. N., 19 S. La Salle St., Chicago, Ill.
*FRIEL, G. M., Anchor Concrete Machy. Co., Columbus, Ohio.
*FRISKE, A. W., Alois, Wis.
*FULLENWIDER, C. V. R., The Philip Carey Co., Lockland, Cincinnati, O.
GALBE, MISS E., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
*GARDINER, LION, Vice-Pres., Lakewood Engineering Co., Cleveland, Ohio.
*GARDINER, W. H., Winston Bros. Company, 801 Globe Bldg., Minneapolis, Minn.
*GARDNER, FRANC J., Atlas Portland Cement Co., 134 S. La Salle St., Chicago, Ill.
*GASTON, H. F., 7945 Eberhardt Ave., Chicago, Ill.
GAUKLER, FRANCIS O., 7237 E. Jefferson Ave., Detroit, Mich.
*GIESECKE, F. E., Engrg. Research Div., University of Texas, Austin, Tex.
*GEORGE, W. J., Alpha Portland Cement Co., 140 S. Dearborn St., Chicago, Ill.
GENUNG, A L., Cal Chemical Co., 327 S. La Salle St., Chicago, Ill.
*GERMUNDSSON, TH., 1130 Wells Bldg., Milwaukee, Wis.
GESSE, D. H., Sanitary Dist. of Chicago, Chicago, Ill.
GILDNER, E. E., Northwestern P. C. Co., Marshalltown, Iowa.
GILDNER, W. E., Northwestern States Portland Cement Co., Demopolis, Ala.
GILLHAM, P. D., Princeton, Ill.
*GINDER, J. W., Supervising Archt., Treasury, Washington, D. C.
*GINSBERG, FRANK I., T. L. Smith Co., 50 Church St., New York City.
*GLEASON, KATE, Commercial St., East Rochester, N. Y.
*GOLDIE, WM. JR., Goldie Mfg. Co., Wilkinsburg Sta., Pittsburgh, Pa.
*GONNERMAN, H. F., Lewis Institute, 1951 W. Madison St., Chicago, Ill.
GOODWIN, P. W., Wing & Evans, Inc., 112 W. Adams St., Chicago, Ill.
GOSSWEIN, O. H., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
*GOULD, HARLEY J., The Ferro-Concrete Constr. Co., Cincinnati, Ohio.
*GOW, CHARLES R., 31 St. James Ave., Boston, Mass.
*GRADY, J. C., Gen. Supt., Turner Constr. Co., 178 Tremont St., Boston, Mass.
GUTHRIE, HENRY A., Portland Cement Association, 111 W. Washington St., Chicago, Ill.
HADLEY, H. M., Dist. Engr., Portland Cement Association, Seattle, Wash.
HAMMOND, C. H., Chatter & Hammond, 64 E. Van Buren St., Chicago, Ill.
HANNA, D. B., Massey Concrete Products Co., 968 Peoples Gas Bldg., Chicago, Ill.
*HANLEY, J. T., American Wire Fence Co., 10 S. La Salle St., Chicago, Ill.
*HANSON, E. S., Box 498, Chicago, Ill.
*HARDING, E. C., Gen. Supt., Ferro-Constr. Co., Cincinnati, O.
*HARKNESS, J. C., The Consolidated Expanded Metal Co., Braddock, Pa.
*HARRIDGE, J. K., Pres., Hydro-Stone Corp., of Ill., 15 E. Van Buren St., Chicago, Ill.
*HARRIS, WALLACE R., 542 Monadnock Blk., Chicago, Ill.
HARRISON, W., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
*HART, W. E., Portland Cement Association, 111 West Washington St., Chicago, Ill.

*HARTY, JOHN J., Monks & Johnson, 99 Chauncy St., Boston, Mass.
*HATT, K. A., F. W. Dodge Corp., 131 N. Franklin St., Chicago, Ill.
*HATT, W. K., Purdue University, Lafayette, Ind.
HATTON, FRANK B., Engineer on Milwaukee Sewerage Comm., Milwaukee, Wis.
*HAVLIK, ROBT. F., Mooseheart, Ill.
*HAWK, LESTER C., Chemist, Dexter Portland Cement Co., Nazareth, Pa.
*HILKER, E. W., Hilker Supply Co., Granite City, Ill.
HELLER, MARTIN, 2439 Madison Ave., Granite City, Ill.
HETHERINGTON, G. F., Marquette Cement Mfg. Co., Chicago, Ill.
HILLMER, JOHN B., 7237 E. Jefferson Ave., Detroit, Mich.
HINKER, J. H., Riley & Hinker, Menominee, Mich.
*HIRSCHBERG, W. P., 218 Stephenson Bldg., Milwaukee, Wis.
*HOLLISTER, S. C., Land Title Bldg., Philadelphia, Pa.
HOLMES, M. E., U. S. Gypsum Co., Chicago, Ill.
HOLMGREN, ROBERT N., State of Ill., Dept. of Public Works, 404 Kimball Bldg., Chicago, Ill.
*HOOK, A. S., Calumet Steel Co., 208 S. La Salle St., Chicago, Ill.
HOVDE, A. D., Capital Ave. Cement Works, St. Paul, Minn.
HOWARD, WM. F., Portland Cement Association, Chicago, Ill.
HUBBARD, C. P., Stone & Webster, Inc., 38 S. Dearborn St., Chicago, Ill.
*HUEBER, PAUL, Hueber Bros., Syracuse, N. Y.
*HUGHES, E. E., Franklin Steel Works, Franklin, Pa.
*HUGHES, T. L., Lehigh Portland Cement Co., 831 Conway Bldg., Chicago, Ill.
*HUMPHREY, D. S., The Humphrey Co., Euclid Beach Park, Cleveland, O.
*HUMPHREY, RICHARD L., 805 Harrison Bldg., Philadelphia, Pa.
HUNLEY, J. B., Bridge Engr., C. C. C. & St. L. Rly., Cincinnati, O.
*HUTCHINSON, G, W. Celite Prod. Co., Raleigh, N. C.
*HUTTER, GEORGE F., Hutter Constr. Co., Fond du Lac, Wis.
ILLSLEY, RICHARD M., 908 Seward St., Evanston, Ill.
*IMLER, ROY F., Thos. W. Noble & Co., 1503 E. 73rd St., Chicago, Ill.
*INGEMANSON, T. W., 5944 W. Erie St., Chicago, Ill.
*INSLEE, C. L., Pres., Guarantee Constr. Co., 140 Cedar St., New York City.
IRWIN, A. C., Portland Cement Assn., Chicago, Ill.
*IRWIN, O. W., 1128 Ford Ave., Youngstown, O.
IVES, C. B., Dixon Concrete Co., 706 E. Fellows St., Dixon, Ill.
IVES, DR. C. H., Dixon Concrete Co., Dixon, Ill.
*JACKSON, C. BLAKE, Pres., Jackson-Lewis Co., Ltd., Federal Bldg., Toronto, Ont.
JACOBSON, H. E., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
JAMES, J. R., Detroit Edison Co., 2000 Second Ave., Detroit, Mich.
JANISCH, JOHN L., Pres. Chicago Cement Burial Vault Co., 5301 Ravenswood Ave., Chicago, Ill.
JENKIN, DANIEL, 801-11 N. Vermillion, Streator, Ill.
JENNINGS, D. F., Associate Editor, *Concrete,* 1807 E. Grand Blvd., Detroit, Mich.
JOHNSON, A. N., University of Maryland, College Page, Md.
JOHNSON, C. H., N. C. & St. L. Ry., Nashville, Tenn.
JOHNSON, MISS E., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
JOHNSON, H. C., Sanitary District of Chicago, Chicago, Ill.
JOHNSON, MARSHALL C., Western Mgr., *Concrete,* 15 E. Van Buren St., Chicago, Ill.
JOHNSON, WM. R., Research Engineer, Portland Cement Association, Chicago, Ill.
*JONES, WALTER E., Federal Concrete Co., 667 Wyoming Ave., Buffalo, N. Y.
JONES, MRS. WALTER E., Buffalo, N. Y.
JOSLIN, OTTO, Supv. Engr., N. C. & St. L. Ry., Nashville, Tenn.
KENDALL, LELAND, Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.

*KAHN, ALBERT, Detroit, Mich.
KAISER, W. G., Portland Cement Association, 111 W. Washington St., Chicago, Ill.
*KALTENBACH, G. W., American Ambi Corp., 345 Madison Ave., New York City.
*KAMMERER CEMENT PRODUCTS CO., El Paso, Ill. M. Kammerer, Supt.
*KELLEY, F. M., Pres., Helderberg Cement Co., Albany, N. Y.
*KELLEY, JOHN M., Henderson Corp., 511 Wood St., Pittsburgh, Pa.
KIENSTRA, FRANK T., Kienstra Bros., Wood River, Ill.
*KINNEY, WM. M., General Manager, Portland Cement Association, Chicago, Ill.
*KING, A. W., Kalman Floor Co., Wrigley Bldg., Chicago, Ill.
*KIRCHER, PAUL, Mgr., Pole Dept., Massey Concrete Products Corp., Chicago, Ill.
*KLEIN, W. H. Dixie Portland Cement Co., Chattanooga, Tenn.
KLEMME, W. G., Hilker Supply Co., Granite City, Ill.
KNEBES, ERVIN L., Asst. City Engineer, City Hall, Milwaukee, Wis.
*KNOWLTON, W. B., American Woolen Co., Andover, Mass.
KOLB, F. X., 1225 E. Grand Blvd., Detroit, Mich.
KRIPPNER, JOHN, 1515 Monroe Bldg., Chicago, Ill.
KROHNE, F. C., Mgr., Preston Products Corp., West Allis, Wis.
*CAST STONE CO., 816 Commerce Bldg., St. Paul, Minn. Harry G. Krum, Secy.
*KVITRUD, I., 754 Builders Exchange, Minneapolis, Minn.
LAWLER, J. G., Universal Portland Cement Co., 210 La Salle St., Chicago, Ill.
LOOMIS, MARCIA, Universal Portland Cement Co., Chicago, Ill.
*LAMBIE, J. E., President, Lambie Concrete House Corp., 5901 Hydraulic Ave., Cleveland, Ohio.
*LAPHAM, JOHN R., George Washington University, Washington, D. C.
*LAPIERRE, C. C., Canada Cement Co., Ltd., Montreal, P. Q., Canada.
LANTZ, S. R., Universal Portland Cement Co., Chicago, Ill.
LEE, J. H., Dist. Engr. of Materials, Spurling Bldg., Elgin, Ill.
*LEFFLER, R. R., 7021 Oriole Ave., Chicago, Ill.
*LEHANEY, J. A., The Kansas Portland Cement Co., Kansas City, Mo.
*LEMOINE, A. B., Canada Cement Co., Ltd., Montreal, P. Q.
*LESSARD, L. P., Kalman Steel Co., Wrigley Bldg., Chicago, Ill.
LIESSMANN, W. A., Elgin Cast Stone Co., 885 St. Charles St., Elgin, Ill.
*LICHTENBERG, E. L., Koehring Co., Milwaukee, Wis.
*LIBBERTON, J. H., 40 Rector St., New York City.
*LINDAHL, O. N., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
*LINDAU, A. E., Otis Bldg., Chicago, Ill. (President American Concrete Institute)
*LINDSLEY, C. E., C. E. Lindsley Co., Irvington, N. J.
LITEHISER, R. R., Portland Cement Association, Chicago, Ill.
*LIVERMORE, A. C., W. A. B. Home Building Co., Wilmerding, Pa.
*LIVINGSTON, J. A., 832 Weston Road, Toronto, Ont.
*LOCKHARDT, W. F., 347 Madison Ave., New York City.
LOEW, OSCAR W., Truscon Steel Co., 7338 Woodward Ave., Detroit, Mich.
*LOFSTEDT, O. B., Modern Construction Co., Grand Junction, Iowa.
*LOCK JOINT PIPE CO., Ampere, N. J. J. E. Longley, Secretary.
*LORD, ARTHUR R., 140 S. Dearborn St., Chicago, Ill.
*LORING, H. D., Chief Engr., Ferro-Concrete Constr. Co., Cincinnati, Ohio.
*LOVE, H. J., Secy., National Slag Assn., Cleveland, Ohio.
*LOVING, M. W., American Concrete Pipe Assn., 111 W. Washington St., Chicago, Ill.
*LOWELL, J. W., Asst. Mgr., Universal Portland Cement Co., Chicago, Ill.
LUDLOW, G. R., Francisco & Jacobus, 511 5th Ave., New York City.
LURIE, E. M., c/o Wharton Clay, 123 W. Madison St., Chicago, Ill.
LUTZ, A., Estimator, C. B. & Q. Rly., Chicago, Ill.
LYNCH, ARTHUR J., *Concrete Products*, 542 Monadnock Bldk., Chicago, Ill.
MOLCAR, CHARLES, Universal Portland Cement Co., Chicago, Ill.

McKoicka, Charles, Downers Grove, Ill.
*Marani, Virgil G., Chief Engr., The Gypsum Industries, 844 Rush St., Chicago, Ill.
MacDonald, H. H., Secy., The Gypsum Industries, 844 Rush St., Chicago, Ill.
*Macgowan, E. S., N. W. Mgr., Service Bureau, Universal Portland Cement Co., Minneapolis, Minn.
Manard, Robert P., W. W. Ahlschlager, Inc., 65 E. Huron St., Chicago, Ill.
*Markland, M. B., Pres., M. B. Markland Co., Atlantic City, N. J.
Markland, N. C., Northwestern Cement Drain Tile Co., Starbuck, Minn.
Marshall, R., *Concrete,* 1807 E. Grand Blvd., Detroit, Mich.
*Martin, E. S., 16 Saulter St., Toronto, Ont.
Martin, S. G., Field Engr., Universal Portland Cement Co., Chicago, Ill.
Mayer, C. H., Schmidt, Garden & Martin, 1515 Monroe Bldg., Chicago, Ill.
*Maynard, Arthur J., State Farm, Mass.
Meaghn, P. H., c/o H. B. Barnard, 140 S. Dearborn St., Chicago, Ill.
*Meier, H. E., N. W. Davenport Cement Block Co., 1419 Division St., Davenport, Ia.
Messinger, F. D., Michigan State Highway Dept., Lansing, Mich.
Metz, Louis A., 3625 Dewey Ave., Omaha, Nebr.
*Meyer, C. Louis, President Concrete Engineering Co., Omaha, Nebr.
*Meyer, M. W., Meyer, Morrison & Co., Inc., 39 Cortlandt St., New York City.
Meyer, R. W., Atlas Portland Cement Co., Corn Exchange Bank Bldg., Chicago, Ill.
*Miller, F. E., The Standard Slag Co., Youngstown, Ohio.
Mohan, M. J., Overland, Mo.
Manson, C. E., Lehigh Portland Cement Co., Kansas City, Mo.
*Moore, O. L., Universal Portland Cement Co., Chicago, Ill.
*Morris, Clyde T., Ohio State University, Columbus, Ohio.
Mowry, C. Wesley, 5724 Maryland Ave., Chicago, Ill.
Mruk, Laudie J., Christofell Concrete Products Co., Milwaukee, Wis.
*McBurney, J. W., 6 Rockwell Bldg., Cleveland, O.
*McCullough, F. M., Carnegie Institute of Technology, Pittsburgh, Pa.
McGraw, George B., Raymond Concrete Pile Co., 111 W. Monroe St., Chicago, Ill.
McKinney, A. J., McKinney-Goodmant Co., Kempton, Ill.
*McKinstry, R. W., 64 N. Monroe St., Chicago, Ill.
*McMillan, E. C., Kalman Floor Co., 508 Empire Bldg., Detroit, Mich.
*McMillan, F. R., 628 Metropolitan Bank Bldg., Minneapolis, Minn.
McMullen, John R., 5057 Lake Park Ave., Chicago, Ill.
McNally, D. H., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
Nagel, Arno, Libertyville Cement Works, Libertyville, Ill.
*Nagaya, S., Chief Engr., Japanese Government Rly., Tokyo, Japan.
*National Lime Assn., Construction Engr., Andrews Hotel, Minneapolis, Minn.
Nelfus, H. V. H., Francisco & Jacobus, 39 S. La Salle St., Chicago, Ill.
Nelson, J. L., Sales Dept., Universal Portland Cement Co., Chicago, Ill.
Nelson, John A., Nelson-Enblom Co., 917 Ply. Bldg., Minneapolis, Minn.
Nevins, Harry G., Salesman, Sandusky Cement Co., Cleveland, O.
Newbold, A. D., Truscon Steel Co., 22 West Monroe St., Chicago, Ill.
*Nichols, Charles E., Stone & Webster, Inc., 147 Milk St., Boston, Mass.
Nichols, Harry W., Geologist, Field Museum, Chicago, Ill.
*Noble, T. W., Thos. W. Noble & Co., 35 S. Dearborn St., Chicago, Ill
Nordstrom, Eben A., Nordstrom & Sons, Rockford, Ill.
Nordstrom, M. V., C. F. Nordstrom & Sons, Rockford, Ill.
Norris, Alvin, 506 9th St., Manitowoc, Wis.
Northway, M. E., Omaha Concrete Stone Co., Omaha, Nebr.
O'Brien, M. J., Underwriter's Laboratories, 207 E. Ohio St., Chicago, Ill.

*OGDEN, WILLIAM, Lakewood Engineering Co., Cleveland, O.
*OLSEN, EUGENE F., Gen. Mgr., Anchor Concrete Machy. Co., Adrian, Mich.
OLSEN, JOHN, President, Anchor Concrete Machy. Co., Rock Rapids, Iowa.
*OHLIGSCHLAGER, J. B., National Concrete Construction Co., 54 Board of Trade, Louisville, Ky.
OHLIGSCHLAGER, CATHERINE, Louisville, Ky.
*OLSEN, E., Badger Concrete Co., Oshkosh, Wis.
OLSEN, GEO. A., Editor, *Building Supply News,* 407 S. Dearborn St., Chicago, Ill.
O'NEIL, H. N., Asst. Sales Mgr., Lehigh Portland Cement Co., 831 Conway Bldg., Chicago, Ill.
*ORD, WILLIAM, 210 N. Clinton St., Chicago, Ill.
*ORNITZ, EDWARD M. (Blaw-Knox Co.), 5734 Wilkins Ave., Pittsburgh. Pa.
OTTE, E. C. (Vice-Pres., American Bldg. Corp.), 5458 Kenmore Ave., Chicago, Ill.
OWENS, L. J., Chief Designer, C. B. & Q. Rly. Co., 547 W. Jackson Blvd., Chicago, Ill.
*PARKIN, C. H., Vortex Mfg. Co., 1978 West 77th St., Cleveland, Ohio.
PAULY, EDW. C., Charles Pauly & Son, Granite City, Ill.
*PEARSON, J. C., Lehigh Portland Cement Co., Allentown, Pa.
*PEASE, B. S., American Steel & Wire Co., 208 S. La Salle St., Chicago, Ill.
*PERRY, J. P. H., Turner Constr. Co., 244 Madison Ave., New York City.
*PETERSON, N. J., Pres., Ideal Cement Stone Co., Omaha, Nebr.
*PEYTON, CHARLES, Marquette Cement Mfg. Co., 140 S. Dearborn St., Chicago, Ill.
*PEYTON, LACY, Peyton's Concrete Works, Benton, Ill.
PINKERTON, EVELYN, Universal Portland Cement Co., Chicago, Ill.
*PIERCE, DANA, President, Underwriters' Laboratories, 207 E. Ohio St., Chicago, Ill.
*PLAYFORD, S., 512 S. Home Ave., Oak Park, Ill.
PORTER, G. W., Universal Portland Cement Co., Chicago, Ill.
PRARD, JOHN G., Gibraltar Cement Block Co., Indianapolis, Ind.
*PRICE, PAUL R., Marquette Cement Mfg. Co., 1331 Marquette Bldg., Chicago, Ill.
*PRIESTER, O. F., Vice-Pres., Priester Constr. Co., Davenport, Iowa.
*PUTMAN, W. B., Asher Fire Proofing Co., 815 15th St., N. W., Washington, D. C.
QUEBBEMAN, EDWARD, Div. Sales Manager, Universal Portland Cement Co., Chicago, Ill.
RAFFIN, LOUIS, 382 East 116th St., Chicago, Ill.
*RANSOHOFF, N., Gen. Mgr., Ideal Concrete Machy. Co., Cincinnati, Ohio.
*RANSOME, A. W., Monadnock Bldg., San Francisco, Calif.
RAWCLIFFE, E. F. (Portland Cement Association), Downers Grove, Ill.
RHOADS, H. SAM, H. Sam Rhoads Co., Taylorville, Ill.
RICE, W. E., Rice & Woods, Tiskilwa, Ill.
*RICHART, F. E., 300 Lab. of Applied Mechanics, Univ. of Ill., Urbana, Ill.
RIDDERWOLD, A. S., Schmidt, Garden & Martin, Monroe Bldg., Chicago, Ill.
ROBERTS, C. A. Galien Concrete Tile Co., Galien, Mich.
ROBERTSON, R. D., Research Engr., 1951 W. Madison St., Chicago, Ill.
*ROBERTSON, W. H., Massey Concrete Products Corp., 969 Peoples Gas Bldg., Chicago, Ill.
ROBINSON, HUGH M., Underwriter's Laboratories, 207 E. Ohio St., Chicago, Ill.
ROBINSON, JAY W., Robinson & Eldridge, Auburn, N. Y.
ROBINSON, L. O., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
ROCKWOOD, NATHAN C., Editor, *Rock Products,* 542 S. Dearborn St., Chicago, Ill.
ROHWER, O. H. D., Universal Portland Cement Co., Chicago, Ill.
ROONEY, A. J., Huron Portland Cement Co., 1325 Ford Bldg., Detroit, Mich.
*ROOS, H. W. The H. W. Roos Co., 2036 Dana Ave., Cincinnati, Ohio.
ROSING, ANTON S., Portland Cement Association, Chicago, Ill.
RUBLE, JOS. S., The H. K. Ferguson Co., Cleveland, Ohio.
RUSH, D. B., Mgr. Cement Dept., Robert W. Hunt Co., 2200 Ins. Exchange Bldg., Chicago, Ill.

RUSSELL, HARRY H., Rice & Woods, Tiskilwa, Ill.
SCHERB, A. W., Universal Portland Cement Co., Chicago, Ill.
SCHAFFER, H. A., Conservation Engr., Portland Cement Association, Chicago, Ill.
*SCHEIDENHELM, EDWARD L., Edward L. Scheidenhelm Co., 111 W. Monroe St., Chicago, Ill.
SCHULTZ, BEN, Elgin Cast Stone Co., Bartlett, Ill.
SCHULTZ, R. E., Elgin Cast Stone Co., Bartlett, Ill.
*SCHURMANN, HOWARD, Independent Concrete Pipe Co., Indianapolis, Ind.
SCHWADA, JOS. P., City Engineer, Milwaukee, Wis.
*SCOTT, S. W., Alpha Portland Cement Co., Easton, Pa.
*SCOTT, W. W., JR., Laclede Steel Co., St. Louis, Mo.
SCOVEL, JAS. D., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
*SEIFERT, FRANK A., Synthetic Tile Co., St. Louis, Mo.
*SHAFER, SAM, JR., Chain Belt Co., Milwaukee, Wis.
SHAW, EDMUND, *Rock Products*, 542 S. Dearborn St., Chicago, Ill.
*SHERTZER, TYRELL B., Ohio Dolomite Association, Dennison Hotel, Columbus, O.
SKOGLUND, J. S., Northwestern Concrete Drain Tile Co., Starbuck, Minn.
*SLATER, W. A., 901 S. Busey Ave., Urbana, Ill.
*SMALL, A. R., Vice-Pres., Underwriters Laboratories, Inc., 207 E. Ohio St., Chicago, Ill.
SMITH, GRANT J., 955 E. 26th St., Erie, Pa.
*SMITH, BLAINE S., Gen. Sales Mgr., Universal Portland Cement Co., Chicago, Ill.
SMITH, CHARLES D., Hutter Constr. Co., Fond du Lac, Wis.
*SNYDER, H. N., Mgr., Buffalo Slag Co., 1072 Ellicott Sq., Buffalo, N. Y.
SNYDER, MRS. H. N., Buffalo, N. Y.
*SORENSON, JULIUS, SONS CO., 1805 W. 6th St., Racine, Wis.
*SPAHR, C. C., 611 Linton Ave., York, Pa. (York Patent Bldr. Blox Co.)
SPEAKES, H. E., President, Speakes Co., St. Paul, Minn.
*SPRINGER, H. B., Gen. Mgr., Dixie Concrete Products Co., Chattanooga, Tenn.
*STARRETT, R. A., Anti-Hydro Waterproofing Co., 317 W. 95th St., New York City.
*STRAUB, F. J., New Kensington, Pa.
*STEPHENS, A. W., Turner Constr. Co., 244 Madison Ave., New York City.
*STEPHANI, E. L., Universal Roofing Tile Co., 4113 N. Keeler Ave., Chicago, Ill.
*STEWARD, CHARLES A., Plano Concrete Works, Plano, Ill.
*STEWARD, W. R., Steward & Silver, 525 E. Hudson, Columbus, O.
*STEWART, C. M., Consolidated Expanded Metal Co., Philadelphia, Pa.
STINEMAN, NORMAN M., Portland Cement Association, Chicago, Ill.
STONE, F. L., Universal Portland Cement Co., Chicago, Ill.
*SWANSON, A. G., Omaha Concrete Stone Co., Omaha, Nebr.
SWETT, LESLIE W., Lakewood Engineering Co., 11 S. La Salle St., Chicago, Ill.
SHRIBER, WM., Universal Portland Cement Co., Chicago, Ill.
*TALBOT, A. N., University of Ill., Urbana, Ill.
*TALBOT, KENNETH H., Koehring Co., Milwaukee, Wis.
TAYLOR, FRANK, Shelbyville, Ill.
TAYLOR, JOS, Portland Cement Association, Chicago, Ill.
TEMPEST, WM. F., Portland Cement Association, Chicago, Ill.
TENCHER, Dist. Manager, Dwight P. Robinson & Co., 1962 Conway Bldg., Chicago, Ill.
TESCH, F. J., Universal Portland Cement Co., Chicago, Ill.
*THOMPSON, SANFORD E., The Thompson & Lichtner Co., 136 Federal St., Boston, Mass.
*THOMSON, W. S., Chief Engineer, Corrugated Bar Co., Inc., Buffalo, N. Y.
THRIFT, B. A., Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
*TINKLER, J. W., 315 Snowden Ave., Toronto, Ont.
*TOCH, MAXIMILIAN, Toch Brothers, 110 E. 42nd St., New York City.

TOLLEN, JOHN, Central Machine Works Co., Minneapolis, Minn.
*TOZZER, ARTHUR C., Turner Constr. Co., 178 Tremont St., Boston, Mass.
TRACY, M. J., Midland Valley Coal & Material Co., Overland, Mo.
TRATMAN, E. E. R., Assoc. Editor, *Engineering News-Record,* 1570 Old Colony Bldg., Chicago, Ill.
*TUBESING, WM. F., Rm. 216, Insurance Bldg., Milwaukee, Wis.
TURNER, J. ARCHER, Turner Constr. Co., 1713 Sansom St., Philadelphia, Pa.
*TURNER, H. C., Turner Constr. Co., 244 Madison Ave., New York City.
*UPSON, MAXWELL M., 140 Cedar St., New York City.
*URQUHART, L. C., Cornell University, Ithaca, N. Y.
*USILTON, L. H., Barney-Ahlers Constr. Co., 110 W. 40th St., New York City.
UTPADEL, ALBERT, Chicago, Illinois.
VAN BENTHUSEN, J. W., Universal Portland Cement Co., Chicago, Ill.
*VAN DOORN, J. C., 836 Security Bldg., Minneapolis, Minn.
VAN ZANT, PAUL C., Cement Securities Co., Denver, Colo.
VOLK, FRED G., Gary Tube Co., 826 Lincoln St., Chicago, Ill.
VENTWEIN, SAM, 308 Bensington, Chicago, Ill.
WILBY, A. C., Universal Portland Cement Co., Chicago, Ill.
WOLF, HAZEL, Universal Portland Cement Co., Chicago, Ill.
*WADDELL, W. H., R. R. No. 2, St. Catherines, Ont., Canada.
WAHL, E. J., The Concrete Stone Silo Co., 514 Masonic Temple, Minneapolis, Minn.
WALKER, E. O., Concrete Engineering Co., Marquette Bldg., Chicago, Ill.
*WALKER, FRANK R., 536 Lake Shore Drive, Chicago, Ill.
WALKER, GEORGE T., Structural Mat. Research Laboratory, Lewis Institute, Chicago, Ill.
*WALKER, STANTON, Lewis Institute, 1951 W. Madison St., Chicago, Ill.
WALLACE, ROBT. W., Universal Portland Cement Co., Chicago, Ill.
*WARD, H. A., Turner Constr. Co., 11 Goodell St., Buffalo, N. Y.
*WARREN, G. E., Portland Cement Association, Chicago, Ill.
*WEITZ, R. W., Charles Weitz' Sons, Des Moines, Iowa.
*WEISS, PHILIP, 132 S. 7th St., Philadelphia, Pa.
*WESTERGAARD, H. M., University of Illinois, Urbana, Ill.
*WETSTEIN, M., Pres., The Concrete Surfacing Machy. Co., 1205 Harrison Ave., Cincinnati, O.
*WHEELER, B. C., Marquette Cement Mfg. Co., 140 S. Dearborn St., Chicago, Ill.
*WHEELER, E. J., Wells Bros. Constr. Co., Chicago, Ill.
*WHIPPLE, HARVEY, *Concrete-Cement Age* Publishing Co., Detroit, Mich.
*WHIPPERMAN, FRANK, Diamond Concrete Products Co., Omaha, Nebr.
*WHITE, W. E., Kalman Steel Co., Chicago, Ill.
WHITING, LEON W., Universal Portland Cement Co., Chicago, Ill.
*WHITNEY, C. S., Hool, Johnson & Whitney, Milwaukee, Wis.
*WIGHT, FRANK C., *Engineering News-Record,* 10th Ave. at 36th St., New York City.
*WIGHTMAN, S. H., 53 W. Jackson Blvd., Chicago, Ill.
WILK, BENJAMIN, Universal Portland Cement Co., Chicago, Ill.
WILLBER, CHARLES E., Willber Concrete Product Co., 234 Sabin St., Jackson, Mich.
WILLIAMS, DAVID, Managing Editor, *Cement and Engineering News,* 22 Quincy St., Chicago, Ill.
WILLIAM, H. R., Materials Service Corp., 21 N. La Salle St., Chicago, Ill.
WILLIAMS, R. A., C. B. Johnson & Son, Conway Bldg., Chicago, Ill.
*WILLIAMSON, F. L., Dewey Portland Cement Co., Kansas City, Mo.
*WILSON, E. B., Pres., American Bureau of Inspection and Tests, Chicago, Ill.
WILSON, J. K., Mgr., Lamar Pipe and Tile Co., 329 Mich. Trust Bldg., Grand Rapids, Mich.
WILSON, RAYMOND, Chemist, 1951 W. Madison St., Chicago, Ill.

*WILLSON, S. H., Kalman Floor Co., 514 Merchants Bank Bldg., St. Paul, Minn.
WITT, G. A., Chicago Manager, Consolidated Exp. Metal Co., 6706 Merrill Ave., Chicago, Ill.
*WITHEY, M. O., Madison, Wis. (University of Wisconsin)
*WITT, J. C., 881 Cornelia Ave., Chicago, Ill.
WOODY, C. C., Secy., Madison Cement Stave Silo Co., Winona Minn.
WOOLDRIDGE, MISS B. E., Universal Portland Cement Co., Chicago, Ill.
*WOOLSON, IRA H., Nat. Board of Fire Underwriters, 76 William St. New York City.
*YANDELL, C. H., Yandell & Conger, Jackson, Tenn.
YOUNG, GEORGE R., 508 Market St., Milwaukee, Wis.
*YOUNG, R. B., Hydro-Electric Power Comm., Toronto, Ont.

MEMBERS AMERICAN CONCRETE INSTITUTE.

An Asterisk (*) designates a Supporting Member.

ABE, MIKISHI, 24 Mita S Temple, Shiba, Tokyo, Japan.

*ABERTHAW COMPANY, 27 School St., Boston, 9, Mass. (L. C. Wason.)

ABRAMS, DUFF A., Lewis Institute, 1951 W. Madison St., Chicago, Ill.

ACHESON, W. M., Div. Engr., New York State Highway Commission, Syracuse, N. Y.

ACME CONCRETE PRODUCTS AND GRAVEL CO., Cement City, Mich. (E. W. Dienhart, Gen. Mgr.)

ADDIS, WILLIAM G., 64 Somerset Ave., Winthrop, Mass.

*AETNA PORTLAND CEMENT CO., 205 Lincoln St., Boston, Mass. (Franklin R. Johnson, Pres.)

AFFLECK, B. F., 210 S. LaSalle St., Chicago, Ill. (Universal Portland Cement Co.)

AGRAMONTE, ALBERT A., Simbron 3191, Buenos Aires, Argentina.

AHEARN, R. L., c/o Barney-Ahlers Constr. Co., 110 W. 40th St., New York City.

AKTIEBOLAGET ARMERAD BETONG, Malmo, Sweden. (David Andersson.)

AKTIEBOLAGET MARINBETONG, Nykoping, Sweden. (E. V. Carlsson.)

ALBRIGHT & MEBUS, 1502 Locust St., Philadelphia, Pa. (Charles F. Mebus.)

ALDRIDGE, E. V., 208 S. LaSalle St., Chicago, Ill. (Universal Portland Cement Co.)

ALGER, H. H., 244 Madison Ave., New York, N. Y. (Turner Constr. Co.)

ALIN, A. L., 2615 Regent St., Berkeley, Calif.

ALLEN, CLEVELAND, 511 5th Ave., New York City. (Francisco & Jacobus.)

ALLEN, LESLIE H., 2137 S. 38th St., Cicero, Ill. (Concrete Tile Machy. Co.)

ALLEN, O. T., 208 S. LaSalle St., Chicago, Ill. (American Steel & Wire Co.)

ALLEN, W. M., SON & CO., 929 Jefferson Bldg., Peoria, Ill. (A. W. Allen, Pres.)

ALLENTOWN PORTLAND CEMENT CO., Allentown, Pa.

ALLIANCE HOLLOW CEMENT BLOCK CO., Northampton, Pa. (J. H. Van Middlesworth.)

ALLISON, H. H., 1015 W. Ash St., Salina, Kans. (Concrete Products Co.)

*ALPHA PORTLAND CEMENT CO., Easton, Pa. (L. Anderson, Jr.)

ALVAREZ, ALBERTO, Calzada Piedad 192, Mexico, D. F., Mex.

ALVAREZ, HNOS, Sucr., Calzada Juarez 35, Queretaro, Qro., Mexico. (Salvador Alvarez.)

AMERICAN AMBI CORP., 345 Madison Ave., New York City. (G. W. Kaltenbach.)

AMERICAN BUREAU OF INSPECTION AND TESTS, THE, Monadnock Blk., Chicago, Ill. (E. B. Wilson, Pres.)

AMERICAN CAN CO., 120 Broadway, New York City. (C. G. Preis.)

AMERICAN CEMENT PAINT CO., Box 294, Chattanooga, Tenn. (W. P. D. Moross.)

AMERICAN CONCRETE CO., 511 Lincoln Bldg., Detroit, Mich. (Stanley H. Edmunds.)

AMERICAN CONCRETE PIPE ASSN., 111 W. Washington St., Chicago, Ill. (M. W. Loving.)

AMERICAN CONCRETE STEEL CO., 27 Clinton St., Newark, N. J. (J. B. Wright.)

AMERICAN HARD WALL PLASTER CO., Utica, N. Y. (T. P. Eldred, Secy.)

AMERICAN SANITARY PRODUCTS CO., Stamford, Conn. (F. M. Brown, Pres.)

AMERICAN SYSTEM OF REINFORCING, 10 LaSalle St., Chicago, Ill. (Arthur A. Clemert.)

ANCHOR CONCRETE MACHINERY CO., 530 Dublin Ave., Columbus, O. (G. M. Friel, Mgr.)

ANCHOR BUILDING CO., 1182 Magnolia Ave., Elizabeth, N. J. (John F. Eilbacher, Pres.)

ANDERSON BROS., 3118 Alamogordo St., El Paso, Tex. (W. R. Anderson.)

ANDERSON, JOHN EDWARD, Box 40, Te Puke, Bay of Plenty, New Zealand.

ANDERSON, W. T., 60 Tennis Place, Forest Hills, Long Island, N. Y.

ANDERSON, WALTER W., Technology Club, of N. Y., 17 Grammercy Park, New York, N. Y.

ANTI-HYDRO WATERPROOFING COMPANY, 39 Cortland St., New York City.

ARAKI, GENJI, Kyoto Imperial University, Kyoto, Japan.

ARBORIO ROAD CONSTRUCTION CO., INC., Main St., Hartford, Conn. (John Arborio, Pres.

ARCHIBALD & HOLMES, Continental Life Bldg., Toronto, Ont. (A. R. Holmes.)

ARNOLD STONE, BRICK & TILE CO., INC., Lem Turner and 47th Sts., Jacksonville, Fla. (M. A. Arnold.)

ARTIFICIAL STONE CO., Bruning, Nebr. (H. R. Park, Mgr.)

ARTSTONE PRODUCTS, INC., 52 Vanderbilt Ave., New York, N. Y. (C. S. Downs.)

ARUNDEL-SHOPE BRICK CO., THE, Pier No. 2, Pratt St., Baltimore, Md. (L. L. Wagner, Pres.)

ASBESTOS SHINGLE, SLATE & SHEATHING CO., Ambler, Pa. (Richard V. Mattison.)

ASHER FIRE PROOFING CO., 915 Southern Bldg., Washington, D. C. (C. B. Asher, Pres.)

ASH GROVE LIME & PORTLAND CEMENT CO., Box 1132, Kansas City, Mo. (A. Lundteigan.)

ASHTON, ERNEST, Allentown, Pa. (Lehigh Portland Cement Co.)

ASSOCIATED METAL LATH MANUFACTURERS, Room 819, 123 W. Madison St., Chicago, Ill. (Wharton Clay.)

ASSOCIATED PORTLAND CEMENT MANUFACTURERS, LTD., Park House, Gravesend, England. (H. R. Cox.)

ASSOCIATION OF PROFESSIONAL ENGINEERS, 406 Fashion Craft Bldg., Winnipeg, Man.

ATWATER, RALPH W., 45 William St., New York City. (McClellan & Junkersfeld.)

AVNSOE, TH., c/o Knickerbocker Portland Cement Co., 342 Madison Ave., New York City.

AYERS, F. C., 250 W. Lafayette Blvd., Detroit, Mich. (Truscon Steel Co.)

BABBITT, ARTHUR B., 268 N. Water St., Kent, Ohio. (The Kent Machine Co.)

BABCOCK, H. A., 1004 Excelsior Life Bldg., Toronto, Ont., Canada.

BACKMAN, L. J., Ferro-Concrete Construction Co., Cincinnati, Ohio.

BADDER, H. CRAIGHILL, 27 Southfield Road, Bedford Park, London W4, England. (The Torbay & Dart Co., Ltd.)

BADGER CONCRETE CO., 191 Marion St., Oshkosh, Wis. (E. Olsen.)

BAER ENGINEERING & CONSTRUCTION CO., Box 74, Leavenworth, Kan. (B. E. Baer.)

BAFFREY, RAYMOND, 1 Rue Danton, Paris, France. (Raymond-Baffrey-Hennebique.)

BAKER, HUGH J., Majestic Bldg., Indianapolis, Ind.

BAKER, JOHN C., 102 N. Beech St., Syracuse, N. Y. (Onondaga Litholite Co.)

BAKER, PHILIP I., 400 Penobscot Bldg., Detroit, Mich. (C. A. P. Turner Co.)

BALDWIN, BERT L., Second National Bank Bldg., Cincinnati, Ohio.

BALLARD, EDWARD H., Chief Engineer Ways and Works, Spencer St., Melbourne, Australia.

BALLINGER, WALTER F., S. E. Cor. 12th and Chestnut Sts., Philadelphia, Pa. (The Ballinger Co.)

BALLOU, HENRY W., 10 Weybosset St., 1035 Grosvenor Bldg., Providence, R. I. (Jenks & Ballou.)

BARBATO, THEODORE, 154 Nassau St., New York City.

BARBER, CHARLES W., 734 Fifteenth St. N. W., Washington, D. C.

BARNARD, IRA B., Box 266, Trinidad, Colo. (The Trinidad Con. Prod. Co.)

BARNEY, W. JOSHUA, 110 W. 40th St., New York City.

BARNEY-AHLERS CONSTRUCTION CORP., 110 W. 40th St., New York City.

*BARNEY-AHLERS CONSTRUCTION CORP., 110 W. 40th St., New York City. (John G. Ahlers.)

BARRIBALL, GEO. D., 13002 Miles Ave., Cleveland, Ohio.

BARRIBALL BROS. CO., THE, 4103 E. 100th St., Cleveland, Ohio. (C. R. Ott, Secy.)

BARROWS, FRANK G., 60 Federal St., Boston, Mass. (National Engineering Co.)

BARTHOLOMEW, TRACY, Mellon Institute, University of Pittsburgh, Pittsburgh, Pa.

BARTLETT, G. S., 208 S. LaSalle St., Chicago, Ill. (Universal Portland Cement Co.)

BARTON, LOREN, 800 Corporation Bldg., Los Angeles, Calif. (Riverside Portland Cement Co.)

BATEMAN, JOHN H., Director, State Highway Laboratory, University of Michigan, Ann Arbor, Mich.

BATE-MCMAHON CO., THE, Ottawa, Ont., Canada. (Engineer and Mgr.)

*BATH PORTLAND CEMENT CO., 1510 Chestnut St., Philadelphia, Pa. (F. Paul Hahnemann.)

BAUER, E. E., 205 Highway Lab., University of Ill., Urbana, Ill.

BAUM, LOUIS, 31 Union Square, New York City.

BAYLISS, FRANK W., 830 State St., Milwaukee, Wis.

BAYLSON, J. JACOB, N. W. Cor. Hope and Norris Sts., Philadelphia, Pa. (Security Insert Co.)

BEARD, ELMER H., 1 Bank St., Room 308 Washington Bldg., Stamford, Conn.

BEARD, FREDERICK S., 2325 Peters Ave., New Orleans, La. (Crescent Wall Plaster Co.)

BEARD, THEODORE W., 71 Wooster, Shelton, Conn. (Waterbury Manufacturing Co.)

BEATTIE & CORNELL, 33 N. Quarry St., Fall River, Mass. (William H. Beattie.)

*BEATTIE, ROY H., 10 Purchase St., Fall River, Mass.

BECKER, WM. C. E., 426 City Hall, St. Louis, Mo. (Chief Engr., Building and Inspection.)

BECKETT, GARNER A., 800 Corporation Bldg., Los Angeles, Calif. (Riverside Portland Cement Co.)

BEELAND, W. J., Macon, Ga.

BEGGS, GEORGE E., Assoc. Prof. Structural Engineering, Princeton, N. J. (Princeton University.)

BEKAY, A. V., 323 Elmwood Ave., Brooklyn, N. Y. (Barney-Ahlers Constr. Corp.)

BELANGER & SON, S., INC., 308 Main St., Nashua, N. H. (Geo. A. Belanger.)

BELL, O. P., The Sunbury Cement Products Co., 300 N. Second St., Sunbury, Pa.

BELWARIAR, S. S., Engineer in Charge, Surma Valley Railway, Project Kadamkuan Road, Patna, India.

BEMIS INDUSTRIES, INC., 40 Central St., Boston, Mass. (A. F. Bemis, Pres.)

BENEDICT, E. L., Union Trust Bldg., Pittsburgh, Pa.

BENESCH, ALFRED, c/o T. P. Rice, 727 Union St., St. Louis, Mo.

BENT BROS., 1714 Eagle St., Los Angeles, Calif. (Arthur Bent.)

BENT, ERNEST F., 419 Grosse Bldg., Los Angeles, Calif. (Bent Concrete Pipe Co.)

BENTHAM, CECIL, 20 Mount St., Manchester, England. (Henry Sims, Ltd.)

BENTLEY, HAROLD, 56a Victoria Bldgs., Manchester, England.

BENTON, WILLIAM, 130 Jones St., Mt. Clemens, Mich. (W. S. Peacock Co.)

BERBERICH, FRANK J., Room 733 Merchants National Bank Bldg., Los Angeles, Calif.

BERGER MFG. CO., Canton, Ohio. (N. P. Wright.)

BERKEBILE BROS., 326 Swank Bldg., Johnstown, Pa. (Louis E. Berkebile.)
BERNACHOT, AUGUSTUS, 710 Green St., Atlantic City, N. J.
BERNIER & SON, N., 411 Walden St., Cambridge, Mass. (Napoleon M. Bernier.)
BERTIN, RENE L., 95 Madison Ave., New York, N. Y. (White Construction Co., Inc.)
BESOSA, J. C. 191 Loiza St., Santurce, Porto Rico.
BEYER & MAHNKEN, Steneck Bldg., Hoboken, N. J. (A. J. Mahnken.)
BIEDERMAN, FRANKLIN, P. O. Box 263, Florence, Ariz.
BILLINGHAM & COBB, 404 Press Bldg., Kalamazoo, Mich. (L. A. Cobb.)
BILLINGS, A. W. K., c/o Brazilian Hydro-Electric Co., Caixa do Carreio No. 2444, Rio de Janiero, Brazil.
BILLINGSLEY CO., THE F. L., 425 Elm St., Cincinnati, Ohio. (Hugh Whittaker, Pres.)
BINDLEY, H. DUNCOMBE, Denton, Harleston, Norfolk, England.
BINSWANGER, S. J., 5520 S. Park Ave., Chicago, Ill.
BICHARD CONSTRUCTION CO., 704 Terminal Bldg., Lincoln, Neb. (B. Sampson.)
BIRMINGHAM SLAG CO., 1607-16 Jefferson County Bank Bldg., Birmingham, Ala.
BIRRELL, DONALD V., 26 Linden St., Brookline, Mass.
B-J CONCRETE PRODUCTS CO., INC., 506 Market St., Red Oak, Ia. (C. H. Barnes, Pres.)
BLACK, ROY, R. F. D. No. 2, Bedford, Ohio.
BLANCHARD, F. E., 75 E. Tompkins St., Columbus, Ohio.
*BLAW-KNOX CO., Box 915, Pittsburgh, Pa. (C. D. McArthur.)
BLISS WATERPROOF CONCRETE BLOCKS, 34 Union Hall St., Jamaica, N. Y. (John J. Bliss.)
BLOECHER, WALTER P., 142 Langdon Ave., Watertown, Mass.
BLUE DIAMOND MATERIALS CO., P. O. Box 1070, Arcade Station, Los Angeles, Calif. (Paul W. Penland, Research Engr.)
BODYCOMB, WALTER C., Box 1909, Birmingham, Ala.
BOGART, MARINE D., Comly Ave., Port Chester, N. Y.
BOLTON PRATT CONTR. CO., 112 Prospect Ave., 801 Columbia Bldg., Cleveland, Ohio. (H. N. Cain.)
BONNELL & SON, J. E., 760 Commerce St., Tacoma, Wash. (C. E. Bonnell.)
BOSCH, JACOB, 100 W. 115th St., Chicago, Ill.
BOUGHTON, WM. HART, 543 8th St., Santa Monica, Calif.
BOUND BROOK CRUSHED STONE CO., 9 Clinton St., Newark, N. J. (L. Upton.)
BOURNE-FULLER COMPANY, THE, 1600 Hanna Bldg., Cleveland, Ohio. (G. C. Smith.)
BOYD, H. H., 5815 Corby St., Omaha, Neb.
BOYER, E. D., 25 Broadway, New York City. (Atlas Portland Cement Co.)
BRADFORD, CHARLES N., 1319 Ardmore Ave., Chicago, Ill.
BRANDT, FRED, P. O. Box 171, La Junta, Colo.

BRANDT, ARTHUR W., 560 Myrtle Ave., Albany, N. Y.
BRASSERT, WALTER, 2006 Race St., Kalamazoo, Mich. (Michigan Silo Co.)
BRAUN, WALTER, 233 S. High St., Columbus, Ohio.
BREED, H. ELTINGE, 507 5th Ave., New York City.
BREIDERT, O. H., 64 E. Van Buren St., Chicago, Ill. (Childs & Smith.)
BREITBEIL, CHARLES E., 103 Johnson St., Lynn, Mass.
BRENNAN, FRANK X, 2507 Tasker St., Philadelphia, Pa.
BRICKETT, EDWARD M., 300 Lab. of Applied Mechanics, Univ. of Illinois, Urbana, Ill.
BRIER HILL CONCRETE PRODUCTS CO., Birmingham, Mich. (Albert V. Schouler.)
*BRITISH COLUMBIA CEMENT CO., LTD., Victoria, B. C., Canada. (Edwin Tomlin, Director-Treas.)
BROMLEY, ALBERT H., 1131 Cleveland Ave., N. W., Canton, Ohio. (Berger Mfg. Co.)
BROWN, E. F., 6332 University Ave., Chicago, Ill.
BROWN, JOHN A. W., 17 Main St., East, Hamilton, Ont. (W. H. Yates Construction Co.)
BROWN, H. WHITTEMORE, Hampton Normal & Agricultural Institute, Hampton, Va.
BROWN, PHILIP B., 410 London Bldg., Vancouver, B. C.
BROWN, REX L., 300 Lab. of Applied Mechanics, Univ. of Ill., Urbana, Ill.
BROWNE, ERNEST, c/o Bero Engineers' Office, Town Hall, Kathrine St., Croydon, W., England.
BROWNING, ALBERT HENRY, 3711 Osgood St., Chicago, Ill.
BRUHN, ELMER F., Box 301, Golden, Colo. (Colo. School of Mines.)
BRUNNER, B. FRANK, 255 Washington St., Royersford, Pa.
BRUNNIER, H. J., 312 Sharon Bldg., San Francis, Calif.
BRYANT, HENRY F., 334 Washington St., Brookline, Mass.
BUERKIN & BUERKIN, 721 Maine St., Quincy, Ill. (Julius A. Buerkin.)
BUFFALO SLAG CO., 1070 Ellicott Sq., Buffalo, N. Y. (H. N. Snyder.)
BUFFALO WASH TRAY WORKS, 601 Tonawanda St., Buffalo, N. Y. (M. J. Deutsch.)
BURDETT, F. A., 25 W. 45th St., New York City.
BURNS, HOMER S., Freeport, Texas. (Freeport Sulphur Co.)
BURROUGHS, H. ROBINS, 70 E. 45th St., New York City.
BURTON, THOMAS E., 1117 Woodrow, Wichita, Kan. (Kansas State Highway Dept.)
BUTLER, D. B., 6 Earl St., Westminster, S. W. 1, London, England.
BUTLER BROS. BLDG. CO., Hamm Bldg., St. Paul, Minn. (Walter Butler.)
BUTLER EQUIPMENT CO., 154 Wisconsin Ave., Waukesha, Wis. (Morgan R. Butler.)
BUTTERFIELD, E. E., 68 Huebers Point Ave., Long Island City, N. Y.
CACCIA, WALTER, 503 W. Union St., Bethlehem, Pa.
CALIFORNIA TYLITE CO., 4707 Hollywood Blvd., Los Angeles, Calif.
*CALUMET STEEL CO., 208 S. LaSalle St., Chicago, Ill. (A. S. Hook.)

CAMPBELL, JOHN T., 1111 Union Bank Bldg., Pittsburgh, Pa. (J. N. Chester, Engineers.)

CAMPBELL, S. A., 251 Elmdorf Ave., Rochester, N. Y.

CAMPION CO., E. S., INC., New Hartford, N. Y. (Edward S. Campion.)

*CANADA CEMENT CO., LTD., Montreal, Que., Canada. (H. S. Van Scoyoc.)

CANADIAN NATIONAL PARKS BRANCH, Dept. of the Interior, Ottawa, Canada. (J. B. Harkin.)

CANON, F. EUGENE, 889 S. 17th St., Newark, N. J.

CAPODAGLI, ALBERTO, R. F. D. 2, Ludlow, Mass.

CAREY CONCRETE CO., 348 Grand Ave., Wisconsin Rapids, Wis. (W. H. Carey.)

*CAREY CO., THE PHILIP, Lockland, Cincinnati, O. (C. V. R. Fullenwider.)

CAREY CO., PHILIP, Lockland, Cincinnati, Ohio. (C. V. R. Fullenwider.)

CARLSON CO., ERNEST F., 244 Main St., Springfield, Mass. (Ernest F. Carlson.)

CARNEGIE LAND CO., 357 Frick Annex, Pittsburgh, Pa. (Roger H. McGee.)

CASE SCHOOL OF APPLIED SCIENCE, Cleveland, Ohio. (F. H. Neff.)

CASPER RANGER CONSTRUCTION CO., 20 Bond St., Holyoke, Mass. (John P. Bathelt.)

CAST STONE CO., 403-4 Commerce Bldg., St. Paul, Minn. (Harry G. Krum.)

CATLETT, RICHARD H., Security Cement & Lime Co., Hagerstown, Md.

CAYE CONSTRUCTION CO., INC., 356 Fulton St., Brooklyn, N. Y. (Webster J. Caye.)

CELITE PRODUCTS CO., 1135 Van Nuys Bldg., Los Angeles, Calif. (R. J. Wig, Vice-Pres.)

CE-LOCK CONCRETE UNITS CORP., 601 Hyde Bldg., Spokane, Wash. (Wm. A. Wells.)

CEMENT & ENGINEERING NEWS, 22 Quincy St., Chicago, Ill. (A. S. Bowes.)

CEMENT-GUN CO., INC., Allentown, Pa.

CEMENT GUN CONST. CO., 537 S. Dearborn St., Chicago, Ill. (J. V. Schaefer, Pres.)

CEMENTILE ROOFING CO., 857 W. 44th St., Norfolk, Va. (A. LeB. Ribble.)

CEMENT PRODUCTS CO., P. O. Box 935, Birmingham, Ala. (G. L. Cox, Pres.)

CEMENT PRODUCTS CO., Winter Haven, Fla. (L. B. Taaffe.)

CEMENT PRODUCTS CO., THE, 207 W. 4th St., Davenport, Ia. (Austin Crabbs.)

CEMENT PRODUCTS CO., Box 123, Wilmington, N. C. (George E. Kidder.)

CHACE, E. M., Room 24, Title Guarantee Bldg., Court and Main Sts., Cincinnati, Ohio.)

*CHAIN BELT CO., 736 Park St., Milwaukee, Wis. (Sam Shafer.)

CHAPMAN, CLOYD M., 171 Madison Ave., New York City.

CHAPMAN, HOWARD, 315 5th Ave., New York City.

CHAPMAN & OXLEY, 506 Harbor Comm. Bldg., Toronto, Ont. (J. Morrow Oxley.)

CHICAGO GRAVEL CO., 1109 Fisher Bldg., Chicago, Ill. (F. W. Renwick, Pres.)

CHITTENDON, HOWARD, E. Main St., Clinton, Conn.

CHRISTENSEN, EINAR, 79 Richmond Ave., Lockpart, N. Y. (Cinder Products Corp.)

CHUBB, JOSEPH, 210 S. LaSalle St., Chicago, Ill. (Universal Portland Cement Co.)

CIAMPA, FELIX A., 101 Park Ave., New York, N. Y. (E. E. Seelye.)

CINCINNATI BUILDERS SUPPLY CO., 534 Main St., Cincinnati, Ohio. (F. H. Kinney.)

CISSEL, JAMES HARLAN, 1213 S. State St., Ann Arbor, Mich. (University of Michigan.)

CLARK, ROBERT J., Hinckley, Ill.

CLARK CEMENT & CONCRETE CONST. CO., Suite 2, Balford Bldg., 634 S. Western Ave., Los Angeles, Calif. (H. K. Koebig.)

CLARKE, STEWART & WOOD CO., 254 Irwin Road, Lexington, Ky. (Chas. O. Wood.)

CLARKE, T. E., Revelstoke, B. C.

CLEMMER, H. F., 100 Washington St., Springfield, Ill.

CLIFFORD & ROEBLAD, 101 Tremont St., Boston, Mass. (Walter W. Clifford.)

*CLINCHFIELD PORTLAND CEMENT CORP., Kingsport, Tenn.

COBB, LOUIS R., 310 N. Fullerton Ave., Montclair, N. J.

COCKEY, J. P., 407 N. Blount St., Raleigh, N. C.

CODDINGTON ENGINEERING CORP., 290 3rd St., Milwaukee, Wis. (Samuel C. Coddington.)

COE, THOMAS H., Box 806, Providence, R. I. (O. D. Purington & Co.)

COHEN, A. B., 1 Madison Ave., New York City.

COLLIER, IRA L., 6019 34th Ave. N. E., Seattle, Wash. (Univ. of Washington.)

COLLINGS CO., W. A., P. O. Box 286, Santa Monica, Calif. (W. A. Collings, Mgr.)

COLMAR, DANIEL, Tivoli St., Albany, N. Y. (Ramloc Stone Co., Inc.)

CONAHEY, GEORGE, Port Norris (Cumb. Co.), N. J. (Field Engr., Portland Cement Association.)

COMMONWEALTH PORTLAND CEMENT CO., LTD., 4 O'Connell St., Sydney, N. S. W., Australia. (Fred J. Beardmore, Secy.)

"CONCRETE," 1807 E. Grand Blvd., Detroit, Mich. (Harvey Whipple, Editor.)

CONCRETE CONSTRUCTION PTY., LTD., 360 Collins St., Melbourne, Australia. (John Gibson.)

CONCRETE PIPE COMPANY, Board of Trade Bldg., Portland, Ore. (Phil Easterday.)

CONCRETE PRODUCTS CO., Finance Bldg., Kansas City, Mo. (E. C. Marqua.)

CONCRETE PRODUCTS CO. OF AMERICA, 805 Diamond Bank Bldg., Pittsburgh, Pa. (C. F. Buente.)

CONCRETE SPECIALTIES CO., P. O. Box 330, Camden, N. J. (L. A. Goodwin, Secy.-Treas.)

*CONCRETE STEEL CO., 42 Broadway, New York City. (John F. Havemeyer.)

*CONCRETE STEEL CO., 42 Broadway, New York City. (Walter S. Edge.)

CONCRETE STEEL BRIDGE CO., 606-626 E. Pike St., Clarksburg, W. Va. (Frank E. McEnteer.)

CONCRETE STEEL ENGINEERING CO., 1904 Park Row Bldg., New York City. (William Mueser.)

CONCRETE STEEL FIREPROOFING CO., 608 Lincoln Bldg., 333 State St., Detroit, Mich.

CONCRETE TILE MACHINERY CO., 2137 S. 48th St., Cicero, Ill.

CONCRETE UNITS CO., Howard and Plummer Sts., Hammond, Ind. (Lawrence Cox, Secy.)

CONDRON CO., 1433 Monadnock Block, Chicago, Ill. (T. L. Condron.)

CONSOLIDATED CONSTR. CO., Durham, N. C. (W. Muirhead.)

*CONSOLIDATED EXPANDED METAL COMPANIES, Pittsburgh District, Braddock, Pa.

CONSTRUCTION MACHINERY CORP., Waterloo, Iowa. (A. M. Locke)

CONSTRUCTION MACHINERY CORP., Waterloo, Iowa. (L. A. Kliebenstein.)

CONWELL & CO., E. L., 2024 Arch St., Philadelphia, Pa. (E. L. Conwell, Pres.)

COOKE, GEO. F., 110 E. 42nd St., New York, N. Y. (Kalman Floor Co.)

COOPER, C. W., Luverne, Ala.

COOPER, G. E., Barnesville, Ohio. (Supt. Cooper Construction Co.)

COOPER, W. R., 231 S. Broad St., Philadelphia, Pa. (Wark Co.)

COPPERSTONE PRODUCTS CO., THE, 220 Albion St., Toledo, Ohio. (A. A. Bennett.)

CORBEN, H. J., City Hall, Darling St., Cape Town, Cape Province, South Africa.

CORBETT, ALEXANDER JOHN, Belleville Hill, Portland, N. S. W., Australia.

CORRIDON, JOSEPH B., 1625 Arch St., Philadelphia, Pa. (Stone & Webster, Inc.)

CORY, RUSSEL G., 50 Church St., New York, N. Y.

COWELL LIME & CEMENT CO., HENRY, 2 Market St., San Francisco, Calif.

COWPER CO., THE JOHN W., Fidelity Bldg., Brooklyn, N. Y. (J. W. Cowper.)

CRAIG-CURTISS CO., THE., 4614 Prospect Ave., Cleveland, Ohio.

*CRAMP & CO., 801 Denckla Bldg., Philadelphia, Pa. (F. L. Durgin, Jr.)

CRANDALL ENGINEERING CO., THE, 102 Border St., East Boston, Mass. (J. Stuart Crandall.)

CRANE, THEODORE, Yale University, New Haven, Conn.

CRANFORD, FREDERICK L., INC., 149 Remsen St., Brooklyn, N. Y. (F. L. Cranford.)

CRARY, ALEX P., 1958 Bogart Ave., Borough of Bronx, N. Y. (Thompson & Binger, Inc.)

CREBE, W. G., 949 Scribner St., Grand Rapids, Mich.

CROOKES, SAMUEL IRWIN, JR., Maungawhau Road, Khyber Pass, Auckland, New Zealand.

CROSS, HARDY, Prof. of Structural Engineering, University of Illinois, Urbana, Ill.

CROWN SIDEWALK & BLOCK CO., INC., 2500 Washington St., N. E., Minneapolis, Minn. (Harry A. Anderson.)

CRUM, R. H., Iowa State Highway Commission, Ames, Iowa.

CUMMINGS, A. E., Room 1912, 111 W. Monroe St., Chicago, Ill. (Raymond Concrete Pile Co.)

CUMMINS, CHARLES A., 243 Calvert Bldg., Baltimore, Md. (Consolidated Engineering Co.)

DAHLBERG, B. G., 645 Michigan Ave., Chicago, Ill. (The Celotex Co.)

DAIDONE, ANTHONY J., 6114 16th Ave., Brooklyn, N. Y. (Atlas Art Stone Works.)

DAMON, ALBERT F., JR., P. O. Bldg., Upper Darby, Del. Co., Pa.

DARLING, E. H. 47 Home Bank Bldg., Hamilton, Ont.

DARUVALA, J. P., Abdulla Bldg., No. 2, Tram Terminus, Parel, Bombay, India.

DAVIDSON, JOHN M., 216 N. 3rd St., Morgan Park, Duluth, Minn.

DAVIS, B. H., 17 Battery Place, New York City. (Bureau of Standards.)

DAVIS, E. E., 1540 Transportation Bldg., Chicago, Ill. (E. E. Davis Co.)

DAVIS-HELLER-PEARCE CO., Delta Bldg., Stockton, Calif. (H. Y. Davis.)

DAVIS, HERBERT A., East New York and Utica Aves., Brooklyn, N. Y. (Brooklyn Crozite Brick Co.)

DAVIS, WATSON, 1442 Rhode Island Ave., N. W., Washington, D. C.

DAVISON, R. GLENN, Jamesburg, N. J.

DAW, E. A. H., Somerset House, 9 Martin Place, Sydney, Australia. (Expanded Steel & Concrete Products Co.)

DAWES, EDWIN ALLEN, c/o Dawes Constr. Supply Co., Thomasville, Ga.

DAWSON, F. M., Lawrence, Kan. (University of Kansas.)

DEEDY, WALTER E., 901 Mills Bldg., El Paso, Texas.

DEFRAIN SAND CO., 804 Finance Bldg., Philadelphia, Pa. (J. L. Durnell.)

DEINBOLL, F. K., 1263 Brookley Ave., Cleveland, Ohio.

DELEUW, C. E., 111 W. Washington St., Chicago, Ill. (Kelker, DeLeuw & Co.)

DELL, ROBERT B., 103 S. 6th St., Duquesne, Pa. (City Engineer.)

DENSMORE, LECLEAR & ROBBINS, 88 Broad St., Boston, Mass. (Henry C. Robbins.)

DENTON & CO., 7 E. 42nd St., New York City. (P. E. Eisenmenger.)

DENTON, W. EDWARD, 3031 K St., N. W., Washington, D. C.

DEPARTMENT OF CIVIL ENGINEERING, University of Texas, Austin, Texas. (Dean T. U. Taylor.)

DEPARTMENT OF THE INTERIOR, San Juan, Porto Rico. (Commissioner Guillermo Esteves.)

DESTREMPS, LOUIS G., 49 Borden Blk., Fall River, Mass.

DETROIT EDISON CO., Detroit, Mich. (A. S. Douglas, Chief Engineer.)
DEWEY, GEORGE T., 613½ Ohio St., Cairo, Ill.
DEWEY PORTLAND CEMENT CO., 409 Scarritt Bldg., Kansas City, Mo. (F. L. Williamson.)
DEXTER PORTLAND CEMENT CO., Nazareth, Pa. (J. B. Brobston.)
DIAMOND CONCRETE PRODUCTS CO., 42nd and Parker Sts., Omaha, Neb. (Frank Whipperman, Pres.)
DICKERSON, JOHN H., 139 Rector St., Elizabeth, N. J.
DINGMAN, CHAS. F., 15 Grove St., Palmer, Mass. (Palmer Constr. Co., Inc.)
DI STASIO, JOSEPH, 120 Liberty St., New York City. (J. Di Stasio & Co.)
DITTMYER, ALEX., 1213 Cora St., Joliet, Ill.
DIVER, M. L., San Antonio, Texas.
DIVISION OF ENGINEERING RESEARCH, University of Texas, Box L, University Station, Austin, Texas. (H. R. Thomas.)
DIXIE CONCRETE PRODUCTS CO., 1010 James Bldg., Chattanooga, Tenn. (H. B. Springer.)
*DIXIE PORTLAND CEMENT CO., Chattanooga, Tenn. (Richard Hardy, Pres.)
DIXON, DEFOREST, 244 Madison Ave., New York, N. Y. (Turner Construction Co.)
DOCKSTADER, ERNEST A., 147 Milk St., Boston, Mass. (Stone & Webster.)
DOELMAN, HERMAN F., 507 N. Charles St., Baltimore, Md.
DOLEMAN, R. E., 311 13th St., N. E., Washington, D. C.
DONOHUE, JERRY, 608 N. 8th St., Sheboygan, Wis.
DOW CHEMICAL CO., THE, Midland, Mich. (S. W. Putnam.)
DOWDELL, C. O., 79 W. Washington St., Chicago, Ill.
DOWNS, ALLAN B., 61 Elm St., Lebanon, N. H.
DOYLE, WILLIAM T., 128 N. Wells St., Chicago, Ill.
DREHMAN PAVING CO., 2622 Parrish St., Philadelphia, Pa. (C. E. Drehman.)
DRESSER-MINTON CO., THE, 1082 The Arcade, Cleveland, O. (J. H. Minton.)
DREW CO., INC., FRED, Woodward Bldg., Washington, D. C. (Fred Drew.)
DREYER, WALTER, 445 Sutter St., San Francisco, Calif. (Pacific Gas & Electric Co.)
DUBUC, N. P., JR., 109 Princeton St., New Bedford, Mass.
DUCKETT, N. S., 12 Castlefield Ave., Toronto, Ont., Canada.
DUNLAP, R. M., 920 Westminster Bldg., Chicago, Ill. (Federal Cement Tile Co.)
DUNN, OSWALD T., 408 N. Prairie St., Champaign, Ill.
DUNNELLS, CLIFFORD G., Head of Dept. of Bldg. Construction, Carnegie Inst. of Technology, Pittsburgh, Pa.
DUNSDON, A. C., c/o East Indian Railway House, 73 King William St., London, E. C. 4, England.
DUPUY, ALBERTO, Apartado 893, Bogota, Colombia, S. A.
DUQUESNE SLAG PRODUCTS CO., Diamond Bank Bldg., Pittsburgh, Pa. (C. L. McKenzie.)

DUTTON, C. B., Monadnock Block, Chicago, Ill. (Besser Sales Co.)
DWYER, JOHN R., Bureau of Standards, Washington, D. C.
EARLEY, JOHN J., 2131 G St., N. W., Washington, D. C.
EASTMAN KODAK CO., Rochester, N. Y. (C. K. Flint.)
*EDISON, THOMAS A., Orange, N. J.
EGBERT, GEORGE W., JR., 1521 Cortelyou Rd., Brooklyn, N. Y.
EGELHOFF, R. F., Turner Constr. Co., 11 Goodell St., Buffalo, N. Y.
EHLERT, E. H., 9119 Harvard Ave., Cleveland, Ohio.
EITZEN, HENRY R., 230 Bay 8th St., Brooklyn, N. Y.
EKHOLM, S. L., 896-902 Farrar St., Cadillac, Mich. (Helm Brick Machine Co.)
ELDRIDGE, H. W., New City, Rockland County, N. Y. (Cement-Gun Co., Inc.)
ELFORD, H., 555 Park St., So., Columbus, Ohio.
ELK RIVER CONCRETE PRODUCTS CO., Elk River, Minn. (D. W. Longfellow.)
ELLOK CORPORATION, 956 Ellicott Sq., Buffalo, N. Y. (James G. Davis.)
EMLEY, WARREN E., 3705 Keokuk St., Washington, D. C. (Bureau of Standards.)
EMMANUELOR, V. E., Bombay Municipality, Office of Works, Love-Grove Road, Work's, Bombay, India.
ENGELMANN, L., Engr., 3644 33rd St., San Diego, Calif.
ENGINEER, R. K., 39 Marine Line, Fort Bombay, India.
ENGINEERING DEPT., City of Rochester, N. Y., 52 City Hall, Rochester, N. Y. (E. H. Walker.)
ENGINEERING NEWS-RECORD, 10th Ave. at 36th St., New York City. (Frank C. Wight.)
ENGLAND, JOHN, Gibbs Chambers, Martin Place, Sydney, N. S. W.
ENGSTROM & CO., 1117 Chapline St., Wheeling, W. Va. (J. W. Wynn.)
ENNIS, JOHN J., N. W. Cor. 3rd and Sycamore Sts., Cincinnati, Ohio. (Jaeger Machine Co.)
ESSELSTYN & CAREY, 603 Hofman Bldg., Detroit, Mich. (H. H. Esselstyn.)
*ETTINGER CONTRACTING CO., INC., 44 Court St., Brooklyn, N. Y. (Louis Ettinger.)
EUPHRAT-HANLEY ESTIMATING & ENGINEERING BUREAU, 305 Walnut St., Cincinnati, Ohio. (Hunter W. Hanley.)
EVANS, FRANK M., 10 W. 28th St., New York, N. Y.
EWING CONCRETE CO., DAVIS, 712 E. Empire St., Bloomginton, Ill. (Davis Ewing, Pres.)
EXPANDED METAL ENGINEERING CO., 525 W. 33rd St., New York City. (W. A. Dittman.)
EYRICK, GEO. J., JR., 800 Marquette Bldg., Detroit, Mich.
FAIR, T. R., P. O. Box 572, Gen. P. O., Calcutta, India.
FAIRBANK BLOCK & SUPPLY CO., LTD., 619 Vaughan Rd., Toronto, Ont. (W. H. Kelk, Secy.)
FAIRCHILD, LEROY F., 3211 Lake Ave., Rochester, N. Y. (Eastman Kodak Co.)

FARMER, HOMER G., Frick Bldg., Pittsburgh, Pa.

FAULKNER, H. F., City Engineer's Dept., Seattle, Wash.

FAULKNER, PROF. F. R., Nova Scotia Technical College, Halifax, N. S.

FAY, FREDERICK R., 200 Devonshire St., Boston, Mass. (Fay, Spofford & Thorndike.)

FEDERAL CEMENT TILE CO., 110 S. Dearborn St., Chicago, Ill. (Leland J. Wilhartz, Secy.)

FEDERAL CONCRETE CO., 667 Wyoming Ave., Buffalo, N. Y. (Walter E. Jones, Secy.)

FELDRAPPE, M. G., 1523 E. 81st St., Cleveland, Ohio. (A. A. Lane Construction Co.)

FERGUSON, JOHN A., 1012 Portland St., Pittsburgh, Pa.

FERGUSON & CO., J. B., Hagerstown, Md. (J. B. Ferguson.)

*FERGUSON CO., JOHN W., United Bank Bldg., 152 Market St., Paterson, N. J. (John W. Ferguson.)

FERGUSON, LEWIS R., 1400 Land Title Bldg., Philadelphia, Pa. (Light, Hollister & Ferguson.)

*FERRO CONCRETE CONSTRUCTION CO., Richmond and Harriet Sts., Cincinnati, Ohio. (W. P. Anderson.)

FERRO CONCRETE CONSTRUCTION CO., Cincinnati, Ohio. (H. D. Loring.)

FERRY CO., INC., JAMES, Virginia and Mediterranean Aves., Atlantic City, N. J. (James V. Ferry.)

FINLAY, L. G., 614 Union Bldg., Cleveland, Ohio. (Raymond Concrete Pile Co.)

FISCHER, JR., ANDREW, 140 Cedar St., New York City. (c/o Walter Kidde & Co., Inc.)

FISCHER, L. J., 51 Wall St., New York City. (Thompson-Starrett Co.)

FISCHER, OTTO F., Norrmalmstorg 3, Stockholm, Sweden. (Betongbyran.)

FISCHER-DEVORE CONSTRUCTION CO., 1025 Dixie Terminal Bldg., Cincinnati, Ohio. (Frank F. Fisher.)

*FISKE-CARTER CONSTRUCTION CO., 11 Foster St., Worcester, Mass. (Burton C. Fiske.)

FLAM, STEPHEN, P. O. Box 265, Huntington Park, Calif.

FLEISCHMANN, LEON, 1650 Broadway, New York City. (Fleischmann Construction Co.)

FLEMING, GEO. S., 120 Madison Ave., Detroit, Mich. (Robert O. Derrick, Inc.)

FLETCHER, AUSTIN B., 2230 Calif. St., N. W., Washington, D. C.

FLETCHER, RICHARD G., 921 15th St., N. W., Washington, D. C. (Fletcher Fireproofing Co.)

FLETCHER-THOMPSON, INC., P. O. Box 85, Bridgeport, Conn. (Edward A. Lambert.)

FOGG, RALPH J., Lehigh University, Bethlehem, Pa.

*FOOTE COMPANY, INC., THE, Nunda, N. Y. (F. L. Dake.)

FORD, MATT, Caldwell, Kan.

FOSTER, ALEXANDER, JR., 6136 Oxford St., Philadelphia, Pa. (William Steel & Sons Co.)

FOUGNER, HERMANN, 103 Park Ave., New York City. (Thompson & Binger, Inc.)

FRANCISCO, F. LEROY, 511 5th Ave., New York City. (Francisco & Jacobus.)

FRANK, HARRY H., 207 Fulton Bldg., Pittsburgh, Pa.

*FRANKLIN STEEL WORKS, Franklin, Pa. (E. E. Hughes.)

FRANKLIN, J., 1218 Wrigley Bldg., Chicago, Ill. (Ideal Concrete Machinery Co.)

FRASER, ALEXANDER, Department of Roads, Quebec, Que.

FREELAND, ROBERTS & CO., 1212 Ind. Life Bldg., Nashville, Tenn. (M. S. Roberts.)

FREEMAN, JOHN E., 122 S. Michigan Ave., Chicago, Ill.

FREEMAN, P. J., 311 Ross St., Pittsburgh, Pa.

FREEMAN, ROGER M., 8 W. 40th St., New York City.

FRENCH, A. W., 202 Russel St., Worcester, Mass. (Worcester Polytechnic Institute.)

FRENCH & CO., S. H., 4th and Callowhill Sts., Philadelphia, Pa. (F. T. McBride.)

FREUND, I. H., 920 Westminster Bldg., Chicago, Ill. (Federal Cement Tile Co.)

FRIDSTEIN, MEYER, 1753 Conway Bldg., Chicago, Ill.

FRIEBELE, J. F., Broad St. Bank Bldg., Trenton, N. J. (Karno-Smith Co.)

FRIEL, FRANCIS S., c/o Albright & Mebus, 1502 Locust St., Philadelphia, Pa.

FRISKE, A. W., Alois P. O., Wis.

FROEHLING & ROBERTSON, Richmond, Va. (H. C. Froehling.)

FROST, CHAMBERLAIN & EDWARDS, Slater Bldg., Worcester, Mass.

FRUCHTBAUM, J., 440 Gurney Bldg., Syracuse, N. Y. (Truscon Steel Co.)

FRUIN-COLNON CONTRACTING CO., 502 Merchants-Laclede Bldg., St. Louis, Mo. (A. P. Greensfelder, Secy.)

FRY, LYNN W., Office of State Architect, Ann Arbor, Mich.

FULLER & MCCLINTOCK, 170 Broadway, New York City. (Geo. W. Fuller.)

FURBER, PIERCE P., Masonic Temple, Danville, Va. (Wiseman & Furber.)

FURLONG, IRVING, 81 Appraiser Bldg., San Francisco, Calif. (Bureau of Standards.)

FUSEJIMA, SHINKURO, Engineering Dept., South Manchuria Railway Co., Ryusan, Chosen, Japan.

GABRIEL STEEL CO., 1150 Penobscot Bldg., Detroit, Mich. (W. F. Zabriskie.)

GALE, L. E., American Trading Co., Hankow, China.

GALIEN CONCRETE TILE CO., Galien, Mich. (C. A. Roberts.)

GARDINER, J. B. W., 30 Church St., Hudson Terminal Bldg., New York, N. Y. (Gardiner & Lewis.)

GARDNER, FRANC E., 3123 Bloomingdale Road, Chicago, Ill. (Gardner-Barada Chemical Co.)

GARDNER, FRANC J., 134 S. La Salle St., Chicago, Ill. (Atlas Portland Cement Co.)

Garties, George, 1100 Telephone Bldg., Cincinnati, Ohio.

Gaskill Constr. Co., 302 Planters Bank Bldg., Wilson, N. C. (W. H. Gaskill.)

Gaston, H. F., 7945 Eberhardt Ave., Chicago, Ill.

Gedney, Ralph, 524 14th St., N. E., Washington, D. C.

Gedney Co., K. H., Hastings, Neb. (Kenneth H. Gedney.)

General Building Co., Inc., 524 Harrison Ave., Boston, Mass. (H. W. Marshall.)

General Fireproofing Co., Youngstown, Ohio. (W. B. Turner.)

Germundsson, Th., 1130 Wells Bldg., Milwaukee, Wis. (Corrugated Bar Co., Inc.)

*Giant Portland Cement Co., Pennsylvania Bldg., Philadelphia, Pa. (Charles F. Conn, Pres.)

Gil, Luis Robles, 9/a Durango, Num. 159 Mexico, D. F., Mexico.

Giles, Allen Lester, 147 Milk St., Boston, Mass. (Stone & Webster.)

Giles, Roy T., 218 New Bern Ave., Raleigh, N. C.

Gill, Grayson, College Station, Texas. (A. & M. College of Texas.)

Gilkey, Prof. Herbert J., Room 212 Engrg. Bldg., University of Colorado, Boulder, Colo.

Gilligan, William H., 31 Union Square, New York City. (Truscon Steel Co.)

Gillis, W. E., Edgerton, Ohio. (Edgerton Cement Works.)

Gilman, Charles, 50 Church St., New York City. (Massey Concrete Products Corp.)

Ginder, J. W., 439 Treasury Bldg., Washington, D. C.

Ginsberg, Frank I., Room 479, 50 Church St., New York, N. Y.

Gleason, Kate, Commercial St., East Rochester, N. Y.

Gleason, Robert W., 634 Madison Ave., Paterson, N. J.

*Glens Falls Portland Cement Co., 205 Lower Warren St., Glens Falls, N. Y. (G. F. Boyle.)

*Glens Falls Portland Cement Co., Glens Falls, N. Y. (Geo. F. Boyle, Jr.)

Glover, Geo. J., 1033 Whitney Bank Bldg., New Orleans, La.

Godfrey, Edward, Monongahela Bank Bldg., Pittsburgh, Pa.

Godley, S. S. and G. H., 716 Southern Railway Bldg., Cincinnati, Ohio. (George H. Godley.)

Goldbeck, A. T., 515 14th St., Washington, D. C. (Bureau of Public Roads.)

Goldie Mfg. Co., Trenton Ave. and P. R. R., Wilkinsburg, Pa. (Wm. Goldie, Jr.)

Gonnerman, H. F., 1951 W. Madison St., Chicago, Ill.

Gottschalk, L. F., Columbus, Neb.

Gould, Harley J., c/o Ferro-Concrete Constr. Co., Cincinnati, Ohio.

Gould Contracting Co., 1214 Ind. Life Bldg., Nashville, Tenn. (C. B. Wilson.)

Gow, Charles R., 80 Boylston St., Boston, Mass.

GRAM, LEWIS M., 912 Oakland Ave., Ann Arbor, Mich. (University of Michigan.)

GRANITE CONCRETE BLOCK CO., LTD., 832 Weston Road, Toronto, Ont. (J. A. Livingston, Pres.)

GRAY CONCRETE CO., Thomasville, N. C. (F. B. Gray.)

GRAY CONSTRUCTION CO., LTD., J. V., 541 Queen St., E., Toronto, Ont. R. J. Fuller.)

GRAY, HOWARD ALLISON, 862 Park Square Bldg., Boston, Mass. (Fay, Spofford & Thorndike.)

GREAT EASTERN GRAVEL CORP., Pt. Jefferson, N. Y. (Geo. D. Perry, Treas.)

GREAT WESTERN PORTLAND CEMENT CO., 410 Land Bank Bldg., Kansas City, Mo. (Page Golsan.)

GREEN, J. SINGLETON, JR., "Ravenshore," Marine Parade Hythe, Kent, England.

GREENE, ROY L., Court House, Chehalis, Wash.

GREEN, VICTOR E., Industrial Research Laboratory, Gas Dept., Council House, Birmingham, England.

GREENMAN, RUSSELL S., State Engineer's Dept., Albany, N. Y.

GREGORY, JULIUS, 49 W. 56th St., New York City.

GRETSCH CONSTRUCTION CO., 50 E. 42nd St., New York City. (Herbert Gretsch.)

GROMFINE, LIEUT. J. J., (CEC) U. S. Navy, U. S. Navy Yard, Puget Sound, Wash.

GUARANTEE CONSTRUCTION CO., 140 Cedar St., New York, N. Y. (Edward Burns.)

GULF CONCRETE PIPE CO., Central Park Station, Houston, Texas. (N. A. Eppes.)

HAGGART, C. N., 331 4th Ave., Pittsburgh, Pa.

HALL, EDWIN C., 1037 45th St., Milwaukee, Wis.

HALL, QUINCY A., 212 Metropolitan Bank Bldg., St. Paul, Minn.

HALL CONSTRUCTION CO., 406 Board of Trade Bldg., Indianapolis, Ind. (R. T. Fatout, Secy.)

HALL & STEVENSON, 409 White Bldg., Seattle, Wash. (J. H. Stevenson.)

HAMILTON, CHARLES T., Powell River, B. C.

HAMMILL, HAROLD B., 42 Portsmouth Road, Piedmont, Oakland, Calif.

HAMMITT, ANDREW B., 31 Union Square, New York City. (Truscon Steel Co.)

HANCOCK, L. W., Law Trust Co. Bldg., Louisville, Ky.

HANNAFORD, H. ELDRIDGE, 1024 Dixie-Terminal Bldg., Cincinnati, Ohio. (S. Hannaford & Sons.)

HANSARD, ORREN H., Tenn. Dept. of Highways and Public Works, Nashville, Tenn.

HANSEN CO., L., 3617 E. 23rd St., Kansas City, Mo. (L. Hansen.)

HANSON, E. S., Box 498, Chicago, Ill. (Assoc. Editor, International Trade Press, Inc.)

HARDING, E. C., Ferro Concrete Construction Co., Cincinnati, Ohio.

HARDY, RICHARD, 1011 James Bldg., Chattanooga, Tenn. (Dixie Portland Cement Co.)

HARESNAPE, V., 800 Corporation Bldg., Los Angeles, Calif. (Riverside Portland Cement Co.)

HARGEN, STANLEY, 133 Rutland Road, Brooklyn, N. Y. (Standard Oil Co.)

HARIG CONSTRUCTION CO., ROBT., 2174 Western Ave., Cincinnati, Ohio. (Ben Harig.)

HARMS, H. J., Rotterdam, Holland. (Continental Petroleum Co.)

HARNISH, JOHN, 737 A. G. Bartlett Bldg., Los Angeles, Calif. (Austin Co., of California.)

HARRIS, C. P., Huron Portland Cement Co., Alpena, Mich.

HARRIS, WALLACE R., 620 Elgin Ave. Forest Park, Ill.

HARRISBURG BUILDING BLOCK CO., Cameron and Reilly Sts., Harrisburg, Pa. (J. Edwin Rutter.)

HARRISON CONSTRUCTION CO., J. S., 2012 Amicable Bldg., Waco, Texas. (C. H. Harrison.)

HART, R. E., 167 Eighth Ave., N., Nashville, Tenn.

HATT, K. A., 131 N. Franklin St., Chicago, Ill.

HATT, WILLIAM KENDRICK, Purdue University, Lafayette, Ind.

HAVLIK, R. F., Mooseheart, Ill.

HAWAIIAN CONTRACTING CO., 854 Kaahumanu St., Honolulu, T. H. (H. P. Benson.)

HAWKINS, J. C., Waterworks Department, City Hall, Cape Town, South Africa.

HAWKINS, PAUL J., 1607 Merchant Bank Bldg., Indianapolis, Ind. (Crawfordsville Foundry Co.)

HAWLEY, JOHN B., 403 Cotton Exchange Bldg., Ft. Worth, Texas.

HAWLEY, WM. H., 2965 Madison Ave., Granite City, Ill.

HAY, WM. WREN, Clinton Ave., R. F. D. 2, Plainfield, N. J.

HAYES, J. E., Engineering Corp., Tientsin, China.

HAYWARD, HARRISON W., Mass. Inst. of Technology, Cambridge, Mass.

HEALY, CLARENCE, Linde-Griffith Co., Newark, N. J.

HEATHER, D. S. B., Land Drainage Branch, Lands and Survey Dept., Pongakawa, Bay of Plenty, New Zealand.

HEIDEMA, P. B., 142 Sip Ave., Jersey City, N. J.

HEINE CHIMNEY CO., 123 W. Madison Ave., Chicago, Ill.

*HELDERBERG CEMENT CO., Albany, N. Y. (Charles R. Parks.)

HELLER-MURRAY CO., 222 W. Rayen St., Youngstown, Ohio. (A. H. Heller.)

HENDERSON CORPORATION, 807 First National Bank Bldg., Pittsburgh, Pa. (Albert Henderson.)

*HERCULES CEMENT CORP., 1600 Walnut St., Philadelphia, Pa. Morris King, Pres.)

HERTZBERG, CHARLES S. L., 239 Confederation Life Bldg., Toronto, Ont. (Harkness, Loudon & Hertzberg.)

HEWES, GEORGE C., 709 W. California St., Urbana, Ill.

HEWETT, W. S., 530 Metropolitan Bank Bldg., Minneapolis, Minn.
HEYWORTH, JAMES O., Harvester Bldg., Chicago, Ill.
HIBBS, MANTON E., 1423 N. 15th St., Philadelphia, Pa.
HILKER SUPPLY CO., 16th and State Sts., Granite City, Ill. (E. W. Hilker.)
HILL, ROGER F., 408 W. Fort St., Detroit, Mich.
HILLAM, A. J., 2280 47th Ave., Oakland, Calif.
HINDMAN, W. S., 3611 Terrace St., Pittsburgh, Pa.
HINTON, GEO. B., Apartado, P. O. Box 60, Mexico, D. F., Mexico.
HIRSCHBERG, WALTER P., 218 Stephenson Bldg., Milwaukee, Wis. (Federal Engineering Co.)
HITCHCOCK, FRANK A., Washington, D. C. (Bureau of Standards.)
HOEFFER & CO., Chamber of Commerce Bldg., Chicago, Ill. (Alexander C. Warren.)
HOFF, J. HAAKON, 208 S. La Salle St., Chicago, Ill. (American Bridge Co.)
HOFF, OLAF, 50 Church St., New York City.
HOLABIRD & ROCHE, 104 S. Michigan Ave., Chicago, Ill. (E. A. Renwick.)
HOLLISTER, S. C., 1400 Land Title Bldg., Philadelphia, Pa.
HOLLOW BUILDING TILE ASSOCIATION, 1409 Conway Bldg., Chicago, Ill. Frank J. Huse.)
HOLMES, FRANCIS, 248 Lambton Quay, Wellington, New Zealand.
HOLT, H. A., Concrete Investigation Dept., 143 Grosvenor Road, London, S. W. 1, England.
HOLTZMAN, S. F., 244 Madison Ave., New York City.
HOOD, RAYMOND M., 40 W. 40th St., New York, N. Y.
HOOL, GEORGE A., College Hills, Madison, Wis. (University of Wisconsin.)
HOOVER, A. P., 150 Janvrin Ave., Bronxville, N. Y.
HOPE ENGINEERING CO., HARRY M., 185 Devonshire St., Boston, Mass. (F. B. Galaher.)
HOPKINS, RALPH Z., 2576 Hurlbut Ave., Detroit, Mich. (Hudson Motor Car Co.)
HORN, A. E., Hancock and Bodine Sts., Long Island City, N. Y. (A. C. Horn Co.)
HORN, H. M., 110 E. 42nd St., New York City.
HORNER, WESLEY W., 300 City Hall, St. Louis, Mo.
HORR, GEORGE E., 244 Madison Ave., New York City. (Turner Construction Co.)
HOUK, HOWARD H., U. S. Bureau of Public Roads, Montgomery, Ala.
HOWE, C. D., The Whelan Bldg., Port Arthur, Ont., Canada.
HOWE, H. N., 76 Porter Bldg., Memphis, Tenn.
HOWES, BENJAMIN A., 70 Fifth Ave., New York City.
HOWES & MCGINTY, INC., 64 Whitman St., New Bedford, Mass. (John J. McGinty.)
HOYER-ELLEFSEN, P. O. Box 463, Kristiania, Norway. (August Gundersen.)
HOYT, W. A., Altoona, Pa.

HUDSON, R. J. H., Public Works Dept., Manchi, India.

HUEBER BROTHERS BUILDERS, INC., 243 Baker Ave., Syracuse, N. Y. (P. J. Hueber.)

HUGGER BROS. CONSTR. CO., Montgomery, Ala.

HUGHES, R. G., 152 Market St., Paterson, N. J. (John W. Ferguson Co.)

HUMBOLDT GRAVEL AND TILE CO., Humboldt, Iowa. (C. L. Douthett, Vice-Pres.)

HUME, ALBERT S., Cangallo 465, Buenos Aires, Argentine.

HUME PIPE CO. (South Africa), Ltd., National Bank Bldgs., Simmonds St., Johannesburg, South Africa. (Walter Wolstenholme.)

HUME, WALTER REGINALD, Reliance House, 301 Flinders Lane, Melbourne, Victoria, Australia, (Hume Pipe Co. of Australia, Ltd.)

HUMPHREY, D. S., Euclid Beach Park, Cleveland, Ohio. (The Humphrey Co.)

HUMPHREY, RICHARD L., 805 Harrison Bldg., Philadelphia, Pa.

HUNT & CO., ROBERT W., 53 Park Place, New York City. (J. F. Davis.)

HUNTER, GEO. H., Santa Barbara, Calif.

HURLBURT, R. W., 100 Jarvis St., Toronto, Ont.

HURLBUT, CHARLES C., 101 Park Ave., New York, N. Y.

*HURON PORTLAND CEMENT CO., 1525 Ford Bldg., Detroit, Mich. (John W. Boardman.)

HUTCHINSON, G. W., Raleigh, N. C. (Eastern Manager, Concrete Dept., Celite Products Co.)

HUTTER CONSTR. CO., 128 Western Ave., Fond du Lac, Wis. (Geo. F. Hutter.)

HYDE, STANLEY T., 212 Ninth St., Bremerton, Wash.

HYDRO-ELECTRIC POWER COMM. OF ONTARIO, 190 University Ave., Toronto, Ont.

IDEAL CEMENT STONE CO., Omaha, Neb. (N. J. Peterson, Pres.)

IDEAL CONCRETE CONST. CO., 455 Rowell Ave., Joliet, Ill. (Gilbert Cooper.)

ILLINOIS STEEL CO., Chicago, Ill. (T. J. Hyman.)

ILLINOIS-WISCONSIN CONCRETE PIPE AND TILE CO., Beloit, Wis. (Chas. E. Richardson.)

IMMEL CONSTRUCTION CO., 98 N. Main St., Fond du Lac, Wis. (Harry W. Mabie, Jr.)

INDEPENDENT CONCRETE PIPE CO., 201 N. West St., Indianapolis, Ind. (Howard Schurmann.)

*INDIANA PORTLAND CEMENT CO., State Life Bldg., Indianapolis, Ind. (Marshal Beck, Treas.)

INDIANA SAND AND GRAVEL PRODUCERS ASS'N., 603 Occidental Bldg., Indianapolis, Ind. (A. M. Brown, Pres.)

INDUSTRIAL ENGINEERING CO., 30 Church St., New York, N. Y. (D. Traver Miller.)

INGBERG, S. H., Bureau of Standards, Washington, D. C.

INGEMANSON, THURE W., 5944 W. Erie St., Austin Sta., Chicago, Ill.

*INLAND STEEL CO., First National Bank Bldg., Chicago, Ill. (G. H. Jones.)

Inness, R. D., c/o Aiken & Inness, Sec. 1 and 2, Welland Ship Canal, St. Catherines, Ont.

Innis, R. L., Govt. Drainage Dept., Thornton, Bay of Plenty, N. Z.

Insley, Wm. H., P. O. Box 167, Indianapolis, Ind. (Insley Mfg. Co.)

*Insley Mfg. Co., P. O. Box 167, Indianapolis, Ind. (Wm. H. Insley.)

*Insley Mfg. Co., P. O. Box 167, Indianapolis, Ind. (Alvin C. Rasmussen.)

Interlocking Cement Stave Silo Co., 709 S. Wichita St., Wichita, Kans. (Kent Merry, Pres.)

*International Cement Corp., 342 Madison Ave., New York City.

Iowa Concrete Products Assn., 405 Hubbell Bldg., Des Moines, Iowa. (Ross Dowell.)

Irwin, Orlando W., 1128 Ford Ave., Youngstown, Ohio. (Truscon Steel Co.)

Irwin & Leighton, 126 N. 12th St., Philadelphia, Pa. (E. M. Campbell.)

Jackson, F. H., U. S. Bureau of Public Roads, Washington, D. C.

Jackson, R. B., 527 W. Ganson St., Jackson, Mich.

Jackson-Lewis Co., Ltd., The, Ryrie Bldg., 229 Yonge St., Toronto, Ont. (C. Blake Jackson, Pres.)

Jacksonville Concrete Products Co., 530 Riverside Ave., Jacksonville, Fla. (Fred C. Hedrick.)

Jacoby, H. S., 6523 Euclid Ave., Cleveland, Ohio. (H. K. Ferguson Co.)

Jeffers, Paul E., 720 Pacific Finance Bldg., Los Angeles, Calif.

Jenrick, Wm. F., 147 Milk St., Boston, Mass.

Jewett, John Y., Administration Bldg., Balboa Park, San Diego, Calif.

Jewkes & Sons Co., Joseph, 676 Montgomery St., Jersey City, N. J. (Francis R. Jewkes.)

Johnson, Algot F., 809 1st National Soo Line Bldg., Minneapolis, Minn.

Johnson, Frank H., Ellsworth, Wis.

Johnson, Lewis J., Harvard University, Cambridge, Mass.

Johnson, N. C., 342 Madison Ave., New York City.

Johnson, T. H., 319 Iowa Bldg., Sioux City, Iowa.

Johnston, Robert S., 3604 McKinley St., N. W., Washington, D. C. (Bureau of Standards.)

Jones, Bevan, 342 Madison Ave., New York City.

Jones, D. W., Supt. of Buildings, City Hall, Binghamton, N. Y.

Jones Construction Co., H. N., Alamo Theater Bldg., San Antonio, Texas. (C. M. Bushick, Vice-Pres.)

Jones, J. Earl, Columbia, N. J.

Jungclaus Co., Wm. P., 825 Massachusetts Ave., Indianapolis, Ind. (F. W. Jungclaus.)

Kahn, Albert, Marquette Bldg., Detroit, Mich.

Kahn, Gustave, Youngstown, Ohio. (Truscon Steel Co.)

Kaiser, B. J., 1515 Union Bank Bldg., Pittsburgh, Pa.

Kalman Floor Co., 410 N. Michigan Ave., Chicago, Ill. (C. E. Cooke.)

*Kalman Steel Co., 410 Michigan Ave., Chicago, Ill. (William S. Thomson.)

*KALMAN STEEL CO., 410 N. Michigan Ave., Chicago, Ill. (A. P. Clark.)

*KANSAS PORTLAND CEMENT CO., Federal Reserve Bank Bldg., Kansas City, Mo. (J. A. Lehaney, Vice-Pres.)

KAPADIA, B. F., Abdulla Bldgs., No. 2, Tram Terminus Parel, Bombay, India.

KAPP, P. B., 707 W. College Ave., State College, Pa. (Penn. State College.)

KATTELLE, WALTER R., Western Electric Co., 110 William St., New York City.

KELLEY, FREDERICK W., 126 State St., Albany, N. Y. (Helderberg Cement Co.)

KELTY, EMER G., 122 N. 51st St., Philadelphia, Pa. (Consolidated Expanded Metal Co.)

KERR, LINTON, 147 Milk St., Boston, Mass.

KIENSTRA BROS. FUEL AND SUPPLY CO., Wood River, Ill. (Frank T. Kienstra.)

KIKUCHI, AITARO, Toa Cement Co., Ltd., Amagasaki, near Osaka, Japan.

KINDLE, GEORGE C., Pitman, N. J.

KING, A. W., 410 N. Michigan Ave., Chicago, Ill. (Kalman Floor Co.)

KINGSBURY, C. T., 216 Woodward Bldg., Washington, D. C. (Rosslyn Steel and Cement Co.)

KINNEY, WILLIAM M., 111 W. Washington St., Chicago, Ill. (Portland Cement Assn.)

KIRK, KARL Q., 520 McCalie Ave., Chattanooga, Tenn.

KITCHEN, R. R. & CO., 802 National Bank Bldg., Wheeling, W. Va. (R. R. Kitchen.)

KLEIN, W. H., Richard City, Tenn. (Dixie Portland Cement Co.)

KLEPACH CONSTRUCTION CO., Cedar Rapids, Iowa. (John Klepach.)

KLINGBERG, W. EARL, 318 Main St., Springfield, Mass.

KLINGER, W. A., Warnock Bldg., Sioux City, Iowa.

KLOCK, MORGAN B., 189 Park Ave., Rochester, N. Y.

KNOPH, OLAF, Prinsensgt 26b, Elevator, Kristiania, Norway.

KNOWLTON, WINFIELD B., 69 Salem St., Andover, Mass.

KNUDSEN, AX. M. AND S. L. SORENSEN, Vesterbrogade 13, Copenhagen, Denmark. (N. J. Nielson.)

KOBER, WM. C., c/o Adensite Co., Inc., 116 W. 39th St., New York City.

*KOEHRING COMPANY, 31st and Concordia Ave., Milwaukee, Wis. (E. H. Lichtenburg.)

*KOEHRING COMPANY, 4940 N. 8th St., Philadelphia, Pa. (P. Koehring.)

KOERNER & CO., C. A., 318 E. Burnett, Louisville, Ky. (R. J. Sweeney.)

KOERNER, CARL A., 426 Odd Fellows Bldg., St. Louis, Mo.

KOHCHI, M., Onoda Cement Co., Yamaguchi-Ken, Japan.

KOMURO, MANGORO, Yotsu Kuracho, Fukushi, Maken, Japan. (Iwaki Cement Co., Ltd.)

KOPITKE, O. F., Wabash and 15th Sts., Toledo, Ohio. (The Gettins Kopitke Co.)

*KOSMOS PORTLAND CEMENT CO., 614 Paul Revere Bldg., Louisville, Ky. (O. N. Clarke.)

KRAUSE, G. E., Juneau, Alaska.

KRAUSE, MARK C., 120 West 4th St., Williamsport, Pa.

KREBS COMPANY, A. J., Walton Bldg., Atlanta, Ga. (A. J. Krebs.)

KRECKER, RAYMOND H., c/o Phila. & Reading Ry., 9th and Spring Garden Sts., Philadelphia, Pa.

KRIER, GEORGE H., 814 E. 94th St., Brooklyn, N. Y.

KUHN, PERRY C., 605 Carondelet Bldg., New Orleans, La.

KVITRUD, I., 754 Builders Exchange, Minneapolis, Minn.

*LACLEDE STEEL CO., 1317 Arcade Bldg., St. Louis, Mo. (W. L. Allen.)

LAGAARD, M. B., Experimental Engineering Bldg., Minneapolis, Minn. (University of Minnesota.)

LAKDAVALA, BURJOR M., c/o H. H. Daruwala, Esq., Outfort, Broach, Bombay, India.

LAKE, SIMON, Milford, Conn.

LAKEWOOD ENGINEERING CO., Cleveland, Ohio. (Lion Gardiner, Vice-Pres.)

LAMB CO., ROBERT E., 843 N. 19th St., Philadelphia, Pa. (Robert E. Lamb.)

LAMBERT, WALTER E., 2028 Lincoln St., Evanston, Ill.

LAMBIE, J. EDWARD, 5901-5999 Hydraulic Ave., Cleveland, Ohio. (Lambie Concrete House Corp.)

LAMBIE, JOSEPH S., Parkman Blvd., Pittsburgh, Pa. (University of Pittsburgh.)

LANCASTER, LIONEL W., 8 McLaren St., Red Bank, N. J.

LANCASTER CONCRETE TILE CO., Lancaster, Pa. (Henry Boettcher.)

LANDER, R. S., Burwell Bldg., Knoxville, Tenn. (Sherman Concrete Pipe Co.)

LANDOR, EDWARD J., 634 Renekert Bldg., Canton, Ohio.

LANE, H. A., Baltimore and Ohio Central Bldg., Baltimore, Md. (Baltimore & Ohio Railroad Co.)

LANG, PHILIP GEORGE, JR., 1300 Baltimore & Ohio Bldg., Baltimore, Md. (Engr. of Bridges.)

LAPHAM, JOHN R., 1829 G St., N. W., Washington, D. C. (George Washington University.)

LARSON, REUBEN LAWRENCE, 4-6 Yuen Ming Yuen Road, Shanghai, China. (Anderson, Meyer & Co., Ltd.)

"LA TOLTECA," Cia de Cemento, Portland, S. A., Independencia 8, P. O. Box 233, Mexico, D. F. Mexico. (G. H. E. Vivian.)

LAVELLE, J., General Assurance Bldg., Bay and Temperance Sts., Toronto, Ont. (Alfred Rogers, Ltd.)

LAVIGNE, ERNEST T., 30 Belvidere Road, Quebec, Que., Canada. (Quebec Provincial Dept. of Public Works & Labor.)

LAZIER, F. S., Welland Ship Canal, Thorold, Ont., Canada.

*LAWRENCE PORTLAND CEMENT CO., 302 Broadway, New York City. (J. S. Van Middlesworth.)

LEA, WILLIAM S., 809 New Birks Bldg., Phillips Square, Montreal, Que. (R. S. & W. S. Lea.)

LEACH, FRED M., 798 Detroit Savings Bank Bldg., Detroit, Mich.
LEAVER, R. J., 49 Swan St., Lawrence, Mass.
LEE, W. HAMILTON, South Plainfield, N. J.
LEEDS & BARNARD, 705 Central Bldg., Los Angeles, Calif. (Chas. T. Leeds.)
LEFFLER, RALPH R., 7021 Oriole Ave., Chicago, Ill.
*LEHIGH PORTLAND CEMENT CO., Allentown, Pa. (H. M. Trexler, Pres.)
*LEHIGH PORTLAND CEMENT CO., Allentown, Pa. (H. M. Trexler, Pres.)
LEHIGH PORTLAND CEMENT CO., Young Bldg., Allentown, Pa.
LENGST, G. J., 212 W. Bluff St., Prairie du Chien, Wis. (Prairie Concrete Prod. Co.)
LEONARD, JOHN B., 57 Post St., San Francisco, Calif.
LEONARD, W. H., 800 Corporation Bldg., Los Angeles, Calif. (Riverside Portland Cement Co.)
LESLEY, ROBERT W., 611 Pennsylvania Bldg., Philadelphia, Pa.
*LEVERING & GARRIGUES CO., 552 W. 23rd St., New York City. (F. S. Wells.)
LEWIS, GEO. H., Malden, W. Va.
*LEY & CO., INC., FRED T., 495 Main St., Springfield, Mass. (Raymond K. Turner.)
LIBBERTON, J. H., 40 Rector St., New York City. (General Chemical Co.)
LIEBERMAN & HEIN, 190 N. State St., Chicago, Ill. (E. Lieberman.)
LILLEY CONCRETE PRODUCTS CO., Aurora, Ill. (L. W. Lilley.)
LIND, PETER & CO., 2 Central Bldg., Westminster, London, S. W. 1, England.
LINDAU, A. E., 10 S. La Salle St., Chicago, Ill. (American System of Reinforcing.)
LINDSAY & CO., W. W., 902 Harrison Bldg., Philadelphia, Pa. (James C. Newlin, Vice-Pres.)
LINDSLEY CO., C. E., 888 Clinton Ave., Irvington, N. J. (C. E. Lindsley.)
LIPSCOMB, P. T., Crockett, Texas.
LINDSTROM, ROBERT S., 203 S. Dearborn St., Chicago, Ill. (Advance Waterproof Cement Co.)
LITTER, F. J., 8 W. 40th St., New York City. (The Frederick Snare Corp.
LIVERMORE, A. C., Mgr. Westinghouse Air Brake Home Bldg. Co., Wilmerding, Pa.
LOCK JOINT PIPE CO., P. O. Box 21, Ampere, N. J. (A. M. Hirsh.)
LOCKE, CLYDE E., 905 Ellicott Square,, Buffalo, N. Y. (A. E. Baxter Eng. Co.)
LOCKWOOD, GREENE & CO., 24 Federal St., Boston, Mass. (Library.)
LOEB, HENRY, II, c/o Loeb Stone Company, Memphis, Tenn.
LOEBER, CHARLES, P. O. Box 1612, Richmond, Va.
LOGEMAN, R. T., 208 S. La Salle St., Chicago, Ill. (American Bridge Co.)
LONEY, NEIL M., 51 Wall St., New York City. (Thompson-Starrett Co.)
LORD, ARTHUR R., 140 S. Dearborn St., Chicago, Ill.

LORENZ Co., P. H., 413 Peoples Bank Bldg., Moline, Ill. (F. R. Dewend.)

LOS ANGELES CONCRETE TILE Co., 432 I. W. Hellman Bldg., Los Angeles, Calif. (Harry Soderberg.)

LOTHIAN, ALBERT J., 230 Chatham St., W., Windsor, Ont., Can.

LOUISVILLE CEMENT Co., 315 Guthrie St., Louisville, Ky. (W. S. Speed, Pres.)

LOVE, HARRY J., 933 Leader-News Bldg., Cleveland, Ohio. (Nat. Slag Assn.)

LOWELL, JOHN W., 208 S. La Salle St., Chicago, Ill. (Universal Portland Cement Co.)

LUETY, GEORGE, 1405 Prairie Ave., Beloit, Wis.

LUNDOFF-BICKNELL Co., THE, 5716 Euclid Ave., Cleveland, Ohio. (C. W. Lundoff.)

LUTEN, DANIEL B., 1056 Lemcke Annex, Indianapolis, Ind. (Luten Engr. Co.)

LYNAM, MAJOR C. G., R. E., Public Works Dept., Bagdad, Mesopotamia.

MACBETH, NORMAN, 800 Corporation Bldg., Los Angeles, Calif. (Riverside Portland Cement Co.)

MCBURNEY, J. W., 6 Rockwell Bldg., Cleveland, Ohio. (Cleveland Board of Education.)

MACDONALD, EARLE, Riverside, Calif. (Riverside Portland Cement Co.)

MCCARTHY, P. A., Box 794, Lufkin, Texas. (Commercial and Industrial Engrg. Co.)

MCCARTHY, T. V., Box 245, Niagara Falls, Ont., Canada.

*MCCLATCHY, JOHN H., 848 Land Title Bldg., Philadelphia, Pa.

MCCLELLAN & JUNKERSFELD, 45 William St., New York City. (H. T. Campion.)

MCCRADY, LOUIS DE B., c/o Canadian Explosives, Ltd., 120 St. James St., Montreal, P. Q., Canada.

MCCULLOUGH, F. M., Carnegie Institute of Technology, Pittsburgh, Pa.

MCDANIEL, ALLEN B., 7 Grafton St., Chevy Chase, Md.

MCEWEN, A. B., c/o Wm. I. Bishop, Ltd., 822 New Birks Bldg., Montreal, P. Q., Canada.

MCHOSE SAND & TILE Co., Boone, Iowa. (Mr. Arthur McHose.)

MCINTYRE, WILLIAM A., 809 Flanders Bldg., Philadelphia, Pa. (Atlas Portland Cement Co.)

MCINTYRE MACHINERY Co., 708 Empire Bldg., Detroit, Mich. (A. E. Carpenter, Secy.)

MCKINSTRY, ROSS W., 205 Kenmore Ave., Elmhurst, Ill.

MCLACHLAN, R. C., c/o J. P. Porter, Standifer & Porter Bros., St. Catherines, Ont.

MCLAUGHLIN & SONS, Mankato, Minn. (J. A. McLaughlin.)

MCLEAN CONTRACTING Co., 1415 Fidelity Bldg., Baltimore, Md. (Oscar B. Coblentz, Pres.)

MCLEAN, WILLIAM K., 8 Spring St., Sydney, New South Wales, Australia.

MCLEOD, WILLIAM, Balgownic Ave., Gonville, Wanganui, N. Z.

MCMILLAN, E. C., 107 Clifford St., Detroit, Mich. (Kalman Floor Co.)

MCMILLAN, FRANKLIN R., 628 Metropolitan Bank Bldg., Minneapolis, Minn. (Shenehon & Meyer.)

MCRAE STEEL CO., 16 McGraw Bldg., Detroit, Mich. (William Corman.)

MCWILLIAM, R. J., "Duart," Hardgrave Road, West End, Brisbane, Australia.

MACATEE, W. L., & SONS, Austin and Commerce Sts., Houston, Texas.

MACK, THOMAS, Peoples Gas Bldg., Chicago, Ill. (Rezilite Mfg. Co.)

MACONI, G. V., 67 Church St., New Haven, Conn. (The Dwight Building Co.)

MAIN, CHARLES T., 200 Devonshire St., Boston, Mass.

MAKI, DR. H., Public Works Bureau, Home Dept. of Japan, Tokyo, Japan.

MALMED, A. T., 1713 Sansom St., Philadelphia, Pa. (A. T. Malmed Co.)

MALONE, JOHN A., Lancaster, Pa. (Malone & Sons.)

MALONEY, ROWLAND, Hyderabad, Deccan, India. (The Reliance Tile Works.)

MANITOBA, UNIVERSITY OF, Sherbrooke and Portage Sts., Winnipeg, Man. (J. N. Finlayson.)

MANTICA, ALBERT J., 301 Journal Bldg., Albany, N. Y.

MARANI, VIRGIL G., 844 Rush St., Chicago, Ill. (Gypsum Industries.)

MARBLE, WILLIAM O., 508 London Bldg., Vancouver, B. C. (Hodgson, King & Marble.)

MARISCAL, FREDERICO E., 9a Colima 292, Mexico City, Mexico.

MARKLAND, M. B., Guarantee Trust Bldg., Atlantic City, N. J.

MARLBORO CEMENT CO., Edmonton, Alberta, Can. (A. W. G. Clark.)

MARQUETTE CEMENT MFG. CO., Marquette Bldg., Chicago, Ill. (T. G. Dickinson.)

MARSCH, LOUIS, Morrisonville, Ill.

MARSH-MURDOCK CO., THE, Melish and Stanton Aves., Cincinnati, Ohio. (George J. Marsh.)

MARSHALL, JOHN, 528 Collins St., Melbourne, Victoria, Australia. (The Marshall Concrete Co., Ltd.)

MARSON, JOHN E., Aurora, Ill. (Barber-Greene Co.)

MARTIN, EDGAR, 104 S. Michigan Ave., Chicago, Ill.

MARTIN, EVAN S., 16 Saulter St., Toronto, Ont. (James A. Wickett, Ltd.)

MASSEY CONCRETE PRODUCTS CORP., Peoples Gas Bldg., Chicago, Ill. (Paul Kircher.)

MASTER BUILDERS CO., THE, 1836 Euclid Ave., Cleveland, Ohio. (S. W. Flesheim.)

MAURO, FRANCESCO, First Nat. Bank Bldg., Birmingham, Ala.

MAYERS, H. WINFIELD, No. 8 Wilson Ave., Watertown, Mass.

MAYNARD, ARTHUR J., Mass. State Farm, State Farm, Mass.

MAYNICKE & FRANKE, 25 Madison Square North, New York City, N. Y. (Julius Franke.)

MAZUR, ISADOR, 3610 Balsam Ave., Indianapolis, Ind.

MEAD, C. A., 165 Wildwood Ave., Upper Montclair, N. J.

MEAD, SUYDAM CO., 342 6th Ave., Newark, N. J. (F. J. Mead, Pres.)
MEDFORD CONCRETE CO., Medford, N. J. (Harry L. King, Jr., Pres.)
MELIN, O. W., Structural Engr., Western Electric Co., Hawthorne Station, Chicago, Ill.
MERCHANT, ARCHIE W., 728 Hospital Trust Bldg., Providence, R. I.
MERLO, MERLO & RAY, LTD., Ford, Ont., Canada. (Louis Alvin Merlo.)
MERRIKEN, C. W., Tacoma Bldg., Chicago, Ill. (Gardiner & Lewis, Inc.)
MERRIMAN, THADDEUS, 2224 Municipal Bldg., New York, N. Y.
MESSEY, LAUREL, Commerce Bldg., Ash and George Sts., Sydney, Australia.
METCALF & EDDY, 14 Beacon St., Boston, Mass. (Frank A. Marston.)
METZGER-RICHARDSON COMPANY, 503 May Bldg., 529 Liberty Ave., Pittsburgh, Pa. (F. L. Metzger.)
MEYER, C. LOUIS, 608 Omaha National Bank Bldg., Omaha, Neb. (Concrete Engrg. Co.)
MEYER, MORRISON & CO., 39 Cortlandt St., New York City. (B. A. Meyer.)
MICHIGAN PORTLAND CEMENT CO., Chelsea, Mich. (G. S. Potter, Jr.)
MICHIGAN UNIVERSITY LIBRARY, Ann Arbor, Mich.
MIDLAND VALLEY COAL & MATERIAL CO., Overland, Mo. (M. J. Mahan.)
MIESENHELDER, P. D., 317 E. 12th St., Indianapolis, Ind. (Indiana State Highway Commission.)
MILBURN LIME & CEMENT CO., LTD., 59 Crawford St., Dunedin, N. Z. (J. H. Stewart, Gen. Mgr.)
MILLER, DANIEL J., Bangor, Pa.
MILLER & SONS' CO., H., 2565 Fifth Ave., Pittsburgh, Pa. (A. G. Miller.)
MILLER, CHARLES R., CO., INC., 556 Susette St., Memphis, Tenn.
MILLER, O. L., & CO., 401 W. 17th St., Indianapolis, Ind. (A. C. Miller.)
MINER, JOSHUA L., 814 Second Place, Plainfield, N. J.
MINSHALL, R. E., 242 S. Gill St., State College, Pa.
*MISSOURI PORTLAND CEMENT CO., Post Dispatch Bldg., St. Louis, Mo. (H. L. Block, Pres.)
MITCHELL, JAMES, 999 Bergen Ave., Jersey City, N. J.
MITCHELL, NOLAN D., 134 Beach St. South, Clarendon, Va. (U. S. Bureau of Standards.)
MODERN CONSTRUCTION CO., Grand Junction, Iowa. (O. B. Lofstedt, Secy.)
MOESER, VICTOR L., Ferro Concrete Construction Co., Cincinnati, Ohio.
MOHLER, JOHN D., Court House, St. Joseph, Mo. (Highway Engineer, Buchanan County.)
MOHR, H. A., 1007 Royster Bldg., Norfolk, Va. (Dist. Mgr. Raymond Concrete Pile Co.)
MOLE, HARRY H., City Engineer, Kearney, Neb.
MOLLENKOF, J. P., c/o John H. McClatchy, Erdenheim, Pa.
MONARCH ENGINEERING CO., Chamber of Commerce Bldg., Buffalo, N. Y. (H. R. Wait, Pres.)
MONKS & JOHNSON, 99 Chauncey St., Boston, Mass. (John J. Harty.)
MONOLITHIC HOLLOW CONCRETE FORM CORP., 326-330 Pacific Finance Bldg., Los Angeles, Calif. (L. J. Desenberg.)

MONTZ, A. S., 707 Title Guarantee Bldg., New Orleans, La.
MOORE, O. L., 1532-210 South LaSalle St., Chicago, Ill.
MOORE, THOMAS, THE ADENSITE CO., 116 W. 39th St., New York City.
MOORES-CONEY CO., 111 E. Fourth St., Cincinnati, O. (W. W. Coney.)
MORE, CHAS. C., Room 305, Education Hall, University of Washington, Seattle, Wash.
MORRILL, F. W., Ferro Concrete Construction Co., Cincinnati, Ohio.
MORRIS, CLYDE T., Ohio State University, Columbus, Ohio.
MORRIS, LLOYD M., 135 S. Atherton St., State College, Pa. (Pennsylvania State College.)
MORRISON, R. L., 4717 First Ave., Birmingham, Ala. (Concrete Products Co.)
MORROW, DAVID W., 4500 Euclid Ave., Cleveland, Ohio.
MORSSEN, C. M., 37 Belmont St., Montreal, Que.
MOSES, FREDERICK W., 10 Weybossett St., Providence, R. I. (Fireman Insurance Co.)
MOTA, CANDELARIO CALOR, 19, 11 de Agosto St., Mayaguez, Porto Rico.
MOYER, ALBERT, 350 Madison Ave., New York City. (Vulcanite Portland Cement Co.)
MUELLER, HAROLD P., 4738 N. Canal St., Logan, Philadelphia, Pa.
MUELLER, J. W., Palladium Bldg., Richmond, Ind.
MUNN, P. J., 147 Milk St., Boston, Mass.
MUNTZ, E. P., 403 Lehigh Valley Terminal, Buffalo, N. Y.
MURPHY, J. C., 714 Louisville Trust Bldg., Louisville, Ky.
MYLCHCREEST, GEO. LEWIS, 238 Palm St., Hartford, Conn. (Buck & Sheldon.)
NAGAYA, S., Chief Engineer, Japanese Government Railway, Tokyo, Japan.
NAITO, TACHU, University of Waseda, Tokyo, Japan. (Engineering College.)
NASH, G. C., P. O. Box 582, Buffalo, N. Y. (Turner Construction Co.)
*NASSAU SAND AND GRAVEL CO., 949 Broadway, New York City. (W. J. Timberman.)
NASU, AKIYA, Kawasaki Works, Nakashibuya, No. 715, Tokyo, Japan.
NATIONAL CONCRETE CONSTRUCTION CO., 54 Bd. of Trade, Louisville, Ky. (J. B. Ohligschlager.)
NATIONAL FIREPROOFING CO., Flatiron Bldg., New York City. (P. Bevier.)
NATIONAL LIME ASSOCIATION, 918 G St., N. W., Washington, D. C. (W. A. Freret.)
NATIONAL LUMBER MFRS. ASSN., 402 Transportation Bldg., Washington, D. C. (D. F. Holtman.)
NATIONAL STONE TILE CORP., 625 Market St., San Francisco, Calif. (C. C. H. Thomas.)
NATSTONE BOSTON CORP., Wellesley Hills, Mass. (Alfred H. Howard.)
*NAZARETH PORTLAND CEMENT CO., Nazareth, Pa. (J. A. Horner.)
NELSON-ENBLOM CO., 917 Plymouth Bldg., Minneapolis, Minn. (Albert Enblom, Secy.-Treas.)

*NEWAYGO PORTLAND CEMENT CO., Newaygo, Mich. (J. D. John.)
NEW JERSEY WIRE CLOTH CO., Trenton, N. J. (Louis G. Beers Manager.)
NEW JERSEY ZINC CO., Palmerton, Pa. (Technical Library.)
NICHOLS, CHARLES ELIOT, 147 Milk St., Boston, Mass. (Stone & Webster, Inc.)
NICHOLS, JOHN R., 45 Milk St., Boston, Mass.
NICHOLSON, JR., JOHN, 2735 Prospect Ave., Cleveland, Ohio.
NOBLE & CO., R. E., 624 Sacramento St., San Francisco, Calif. (Theodore P. Dresser, Jr.)
NOBLE, THOMAS W., & CO., 35 S. Dearborn St., Chicago, Ill. (T. W. Noble, Gen. Manager.)
NOICE, BLAINE, 1326 Washington Bldg., Los Angeles, Calif.
NOONAN, W. H., Metropole Bldg., Halifax, Nova Scotia.
N. W. DAVENPORT CEMENT BLOCK CO., 1725 Davie St., Davenport, Iowa. (H. E. Meier.)
NORTHWESTERN STATES PORTLAND CEMENT CO., Mason City, Iowa. (W. Cowhan.)
NOVELLA, GUSTAVO, Avenida del Hipodromo, Guatemala, Guatemala, C. A.
OAKEY, CHARLES W., 915 Second Ave., Eau Claire, Wis.
O'CONNELL, SIMON T., 237 Darragh St., Pittsburgh, Pa.
OEHMANN, JOHN W., Room 110, District Bldg., Washington, D. C. (Prin. Asst. Inspector of Buildings, D. C.)
OEHRLE, WILLIAM, 342 Madison Ave., New York, N. Y.
OESTERBLOM, I., The Truscon Steel Co., Examiner Press Bldg., Meadows St., Bombay, India.
OGDEN PORTLAND CEMENT CO., Room 521, Eccles Bldg., Ogden, Utah. (R. C. Briscoe.)
OGDEN, WILLIAM, Madison, Ind. (Rep., Lakewood Engineering Co.)
OKUBO, TOSHIYUKI (Truscon Steel Co. of Japan), Yurakucho Kojimachi, Tokyo, Japan.
OLSON, OLE K., 833 Pendido St., New Orleans, La.
ORD, WILLIAM, 210 North Clinton St., Chicago, Ill.
ORNITZ, EDW. M., 5734 Wilkins Ave., Pittsburgh, Pa.
ORR, JOHN B., 6th St., Miami, Fla.
OSCAR, L. C., Bureau of Standards, Washington, D. C.
O'SHEA, D. M., c/o H. S. Ferguson, Consulting Engr., 200 Fifth Ave., New York City.
*OTTAWA SILICA CO., Ottawa, Ill. (C. B. Herring.)
OVERPECK, O. E., Swift & Co., Chicago, Ill.
PACIFIC STONE CO., 4257-8 N. W., Seattle, Wash. (A. W. Swartz, Mgr.)
PALMER CONCRETE PRODUCTS CO., 171 Lowell St., Peabody, Mass. (Osborn Palmer, Gen. Mgr.)
PANZER, R. R., 609 Southern Ohio Bank Bldg., Cincinnati, Ohio.
PARKER, FRANK S., 280 Madison Ave., New York City. (Parker & Shaffer.)
PARRY, CHARLES, 510 Pennsylvania Bldg., Philadelphia, Pa. (Chief Engr. Direct Oxidation Disposal Co.)

PATEL, N. T., Masland-Price & Co., Nesbit Road, Mazagaon, Bombay, India.

PATTILLO, JAMES N., 1716 W. 42nd St., Los Angeles, Calif.

PATZIG, MONROE L., 206 Eleventh St., Des Moines, Iowa.

PEABODY, DEAN, JR., Room 301, Mass. Institute of Technology, Cambridge, Mass.

PEARSE, LANGDON, 910 S. Michigan Ave., Chicago, Ill. (Sanitary District of Chicago.)

PEARSON, J. C., Young Bldg., Allentown, Pa. (Lehigh Portland Cement Co.)

PEASE, B. S., 208 S. LaSalle St., Chicago, Ill. (Am. Steel & Wire Co.)

PEDEN, L. T., P. O. Box 341, Houston, Texas.

PEERLESS CONCRETE PRODUCTS CO., 4956 8th Ave., S., Seattle, Wash.

*PEERLESS PORTLAND CEMENT CO., Union City, Mich. (Wm. M. Hatch.)

*PENN-ALLEN PORTLAND CEMENT CO., Widener Bldg., Allentown, Pa. (W. E. Eidel.)

PENN BUILDING BLOCK CO., INC., 923 Sansom St., Philadelphia, Pa. (Raymond M. Weeks.)

*PENNSYLVANIA CEMENT CO., 131 E. 16th St., New York City. (William Beach.)

*PENNSYLVANIA CEMENT CO., 131 E. 16th St., New York, N. Y. (W. N. Beach, Pres.)

PENNSYLVANIA STATE HIGHWAY DEPT., Harrisburg, Pa. (W. H. Connell, Asst. State Highway Comm.)

PERKINS, RUPERT G., c/o H. S. Ferguson, Consulting Engr., 200 Fifth Ave., New York City.

PERROT, EMILE G., Boyertown Bldg., 1211 Arch St., Philadelphia, Pa.

PERROTT, LESLIE M., Architect, 243 Collins St., Melbourne, Victoria, Australia.

PERRY, BRIAN R., 404 New Birks Bldg., Montreal, Canada. (MacKinnon Steel Co., Limited.)

PERRY, J. P. H., Vice-President, Turner Construction Co., 244 Madison Ave., New York City.

PERRY, L. A., Longview, Wash. (The Longview Co.)

PEYTON, LACY, Benton, Ill. (Peyton's Concrete Works.)

*PHOENIX PORTLAND CEMENT CO., Age-Herald Bldg., Birmingham, Ala. (R. E. Roscoe, Chief Chemist.)

PIERSON, CHARLES V., 202 S. 21st Ave., West, Duluth, Minn. (Duluth Bldrs. Supply Co.)

PIERSON CONCRETE PRODUCTS CO., 89 Dodd St., East Orange, N. J. (James T. Pierson, Vice-Pres.)

PIGOTT, JOSEPH M., Pigott-Healy Construction Co., Hamilton, Ont., Can.

PINNELL, JAS., 90 West St., New York City. (Rollin Constr. Corp.)

PITTSBURGH BLDG. SPEC. CO., 311 Ross St., Pittsburgh, Pa.

PITTSBURGH TESTING LABORATORY, 7th and Bedford Ave., Pittsburgh, Pa. (F. R. Rood.)

PLAGWIT, ERIC, Room 312, 311 Ross St., Pittsburgh, Pa.

PLANO CONCRETE WORKS, Box B, Plano, Ill. (Chas. A. Steward.)
PLUMER, H. E., 22 Ellicott Square, Buffalo, N. Y.
POLARIS CONCRETE PROD. CO., Box 86, W. Duluth, Minn. (E. H. Dresser, Pres.)
THE POLLAK STEEL CO., P. O. Box 1461, Cincinnati, Ohio. (Julian A. Pollak, Vice-President.)
PONS, FRANCISCO, Santurce (nr. San Juan), P. R.
PORTER, J. M., Easton, Pa.
POST & MCCORD, 101 Park Ave., New York, N. Y. (Andrew J. Post, Pres.)
POWELL, FRANCIS C., 64 Pleasant St., Dorchester, Mass.
POWELL, F. E., 25-27 Ferry Bldg., Auckland, N. Z.
POWERS & SON, EUGENE S., 315 S. 15th St., Philadelphia, Pa. (E. S. Powers.)
POWERS KENNEDY CONTRACTING CORP., 149 Broadway, New York City. (George C. Bingham.)
POZZO, ALBERTO, Corso Re Umberto 63, Torino, Italy.
PRACK, BERNARD H., 50 Bay St., Toronto, Ont. (Prack & Perrin.)
PRATT, H. B., Antrim, N. H.
PRIESTER CONSTR. CO., 1006 Kahl Bldg., Davenport, Iowa. (O. F. Priester.)
PRITCHELL, FRANK S., Concrete Laboratory, Raleigh, N. C. (N. C. State Highway Commission.)
PUCCI, ANGELO, 325 Central Park, Rochester, N. Y.
PURVES, JOHN, 914 Monadnock Block, Chicago, Ill. (Wells Bros. Const. Co.)
QUEBEC DEPT. OF PUBLIC WORKS & LABOR, Government Bldgs., Quebec, P. Q. (Ivan E. Vallee.)
QUEBEC DEPT. OF ROADS, Parliament Bldgs., Quebec, P. Q. (J. L. Baulanger.)
RABBERS, OSCAR A., 1619 Reed Ave., Kalamazoo, Mich.
RABER & LANG MFG. CO., Kendallville, Ind. (John E. Lang, Pres.)
RADER, B. H., Conway Bldg., Chicago, Ill. (Lehigh Portland Cement Co.)
RADIGAN, FRANK J., Court House, Jersey City, N. J.
RANDALL, FRANK A., 160 N. LaSalle St., Chicago, Ill.
RANSOHOFF, V., Ideal Concrete Machine Co., Cincinnati, Ohio.
*RANSOME CONCRETE MACHINERY CO., Dunellen, N. J. (A. P. Robinson.)
*RAYMOND CONCRETE PILE CO., 140 Cedar St., New York City. (H. P. Hamlin.)
RAYWID, LEO., 719 Quebec Place, N. W., Washington, D. C.
REBELLEDO, MIGUEL, 11a Artes 169, Mexico City, D. F., Mexico.
REED CO., WILLIAM T., 200 Devonshire St., Boston, Mass. (William T. Reed.)
RENO CONCRETE FACTORY, 1026 W. 1st St., Reno, Nevada. (L. C. Bernasconi.)
REYNVAAN, A. J., 2036 Wendover, Pittsburgh, Pa.
RHEINSTEIN & HASS, INC., 21 E. 40th St., New York City. (A. Rheinstein.)

Rhett, Albert H., 8 Hillside Ave., Summit, N. J.

Rib-Stone Concrete Corp., 2-3 Chamber of Commerce Bldg., Batavia, N. Y. (George E. Priest.)

Rice, James, P. O. Box 10, Forest Hills, L. I.

Rice, John A., 1165 Arch St., Berkeley, Calif.

Rich, Melvin S., 1410 H St., N. W., Washington, D. C.

Richards II, Clarence E., 584 E. Broad St., Columbus, Ohio. (Richards, McCarty & Bulford.)

Richardson, James H., Technology Club, 17 Gramercy Park, New York City.

Richardson-Jones, Harry, 1534 Magazine St., Honolulu, Hawaii.

Richart, Frank E., University of Illinois, 300 Lab. of Applied Mechanics, Urbana, Ill.

Richmond, Knight C., 10 Weybossett St., Providence, R. I.

Riegeluth, Ray J., 58 Waverly St., New Haven, Conn.

Riesche, Robert H., 306 Trimble Blk., Sioux City, Iowa. (Riesche & Sanborn.)

Ritter, Louis E., 140 S. Dearborn St., Chicago, Ill. (Ritter & Matt.)

River Grip Steel Co., 2735 Prospect Ave., Cleveland, Ohio. (G. G. Grenlich.)

Roberts & Schaefer Co., 1110 Wrigley Bldg., Chicago, Ill. (E. E. Barrett, Vice-Pres.)

Robinson, Albert Fowler, Room 1033, Railway Exchange Bldg., Chicago, Ill. (A. T. & S. F. R. R. System.)

Robinson, C. C., 1004-5 Times-Despatch Bldg., Richmond, Va. (Chas. M. Robinson.)

Robinson & Co., Inc., Dwight P., 125 E. 46th St., New York City. (M. E. Thomas.)

Rock Products, 906, 542 S. Dearborn St., Chicago, Ill. (Edmund Shaw.)

Rockefeller, Lloyd H., Westmoreland St. Wharves, Philadelphia, Pa. (Pa. Brick & Tile Co.)

Rogers, Floyd, Newton, Iowa.

Roos Co., The H. W., 2036-46 Dana Ave., Cincinnati, Ohio. (H. W. Roos.)

Roos-Meyer-Hecht Co., The, 2814 Stanton Ave., Cincinnati, Ohio. (G. W. Meyer, Secy.)

Root, Col. Wm. H., Park Square Bldg., Boston, Mass. (319th Engrs. 94th Div.)

Rorick, C. L., 133 W. Washington St., Chicago, Ill. (The Permanent Bldr., Inc.)

Rosholt Co., Thorman W., 421 S. 5th St., Minneapolis, Minn. (Thorman W. Rosholt.)

Ross, Frank B., 310 6th St., Laurel, Miss.

Rowell, W. A., Lakeport, N. H.

Royal Swedish Board of Waterfalls, Regeringsgatan 45, Stockholm, 3, Sweden. (Axel Ekwall.)

RUSSELL, H. M., Riverside, Calif. (Riverside Portland Cement Co.)
RUST ENGRG. CO., 311 Ross St., Pittsburgh, Pa. (T. H. Wincherte.)
RYAN, WILLIAM R., 49 Wall St., New York City. (Thompson-Starrett Co.)
*ST. MARY'S CEMENT CO., LTD., Room 14, 49 Wellington St., East, Toronto, Ont. (Geo. H. Gooderham, Pres.)
ST. PAUL CEMENT WORKS, 34 E. 4th St., St. Paul, Minn. (H. C. Berchem.)
SAMPSON, GEORGE A., 83 Pembroke St., Newton, Mass. (Weston & Sampson.)
SANDQUIST & SNOW, INC., 323 Calumet Bldg., Miami, Fla. (Welton A. Snow, Chief Engineer.)
SANDERSON & PORTER, 52 William St., New York, N. Y.
SANDSTROM, CHARLES O., 2931 Campbell St., Kansas City, Mo.
*SANDUSKY CEMENT CO., 626 Engineers Bldg., Cleveland, Ohio. (William B. Newberry.)
SANTA CRUZ PORTLAND CEMENT CO., Davenport, Calif. (Llewellyn T. Backman.)
SAUM, IRVING R., 3218 Newark St., N. W., Washington, D. C. (Wardman Constr. Co., Inc.)
SAUNDERS, W. L., 1812 G St., Washington, D. C. (Concrete Steel Co.)
SAURBREY, ALEXIS, 2112 Oliver Bldg., Pittsburgh, Pa. (Mellen-Stuart Co.)
SAVILLE, CHRISTOPHER JAMES, Portland, New South Wales, Australia. (Commonwealth Portland Cement Co.)
SCHEINDENHELM CO., EDWARD L., 111 W. Monroe St., Chicago, Ill. (Edward L. Scheidenhelm.)
SCHLYTER, RAGNAR, State Testing Institute, "Statens Provningsanstalt," Stockholm, Sweden.
SCHMIDT, PAUL S., 4500 Euclid Ave., Cleveland, Ohio.
SCHNACK, BENNO J., Calle Mariano Pelliza, 886 Olivos, Buenos Aires, Argentina Republic.
SCHOENTAG, DAVID, INC., 68 Ulster Ave., Saugerties, N. Y. (David Schoentag, Pres.)
SCHOFIELD, R. W., Whakatone, New Zealand.
SCHOLER, PROF. CHAS. H., Dept. of Applied Mechanics, Kansas State Agricultural College, Manhattan, Kans.
SCHOULER CEMENT CONSTRUCTION CO., 154-6 Frelinghuysen Ave., Newark, N. J. (D. D. Schouler.)
SCHUSTER, K. R., 15 Park Row, New York, N. Y.
SCHWADA, JOSEPH P., City Engineers' Office, City Hall, Milwaukee, Wis.
SCHWALBE, WILLIAM, 711 W. Springfield St., Urbana, Ill. (University of Illinois.)
SCOFIELD ENGR. CONSTRUCTION CO., Pacific-Finance Bldg., Los Angeles, Calif. (C. M. Scofield.)
SCOTT, J. R., Ross & Scott, Welland, Ont., Can.
SEARL, THOMAS D., 949 Broadway, New York City. (Geo. A. Fuller Co.)

Security Cement & Lime Co., Hagerstown, Md. (John Porter.)
*Security Cement & Lime Co., Citizens Bank Bldg., Baltimore, Md. (Loring A. Cover, Pres.)
Seelye, Elwyn E., 101 Park Ave., New York City.
Seifert, Frank A., 2627 Dayton St., St. Louis, Mo. (Synthetic Tile Co.)
Semon, Edward G., 721 Chrisler Ave., Schenectady, N. Y.
Setna, J. N., c/o Messrs. Thomas Cook & Sons, Bombay, India.
Sewell, John S., c/o Alabama Marble Co., Birmingham, Ala.
Sexton, F. H., N. S. Technical College, Halifax, N. S.
Shaffer, Ivan O., 280 Madison Ave., New York City. (Parker & Shaffer.)
Shank, J. R., 97 W. Tompkins St., Columbus, Ohio.
Shattuck, Inc., L. H., 208 Granite St., Manchester, N. H. (George G. Shedd.)
Sheldon, F. P. & Sons, Hospital Trust Bldg., Providence, R. I.
*Shenk Co., Henry, Erie, Pa. (E. R. Shenk.)
Sherman, Ralph A., Quarry and Taylor Place, Trenton, N. J. (State Highway Comm. Laboratory.)
Shertzer, Tyrill B., Dennison Hotel, Columbus, O. (Engr. Mgr., Ohio Dolomite Assn.)
Shibaura Engineering Works, Ltd., 1 Shinhamacho, Kanasugi, Shiba-Ku, Tokyo, Japan. (M. Sekiguchi.)
Shoemaker, Marshall N., 15 Central Ave., Newark, N. J.
Shope Brick Co., 361½ E. Morrison St., Portland, Ore. (D. F. Shope.)
Siesel, S. M., 105 Wells St., Milwaukee, Wis.
*Signal Mountain Portland Cement Co., Chattanooga, Tenn. (John L. Senior, Pres.)
Simpson, Alex., Jr., Co., Colo. National Bank Bldg., Denver, Colo. (Alex. Simpson, Jr.)
Simpson, George, 51 Wall St., New York City. (Thompson-Starrett Co.)
Simpson, John M., 230 Border St., East Boston, Mass.
Simpson, Louis, 172 O'Connor St., Ottawa, Ont.
Simpson Bros. Corp., 166 Devonshire St., Boston, Mass. (J. P. Simpson.)
Sinclair & Grigg, 401 Medical Arts Bldg., Philadelphia, Pa.
Siochi, Pedro, Salazar, 407, Manila, P. I.
Skay, A. J., 3925 Nebraska Ave., St. Louis, Mo.
Slater, Willis A., 809 S. Busey St., Urbana, Ill. (Bureau of Standards.)
Slover, Edward, Camden, Ohio.
Smallwood, L. C., St. Elmo Sta., Chattanooga, Tenn.
Smith, Blaine S., 210 S. La Salle St., Chicago, Ill. (Universal Portland Cement Co.)
Smith, George A., Bureau of Standards, Washington, D. C.
Smith, Grant J., 955 East 26th St., Erie, Pa.
Smith, Major N. H., Imber Court, East Molesey, Surrey, England.
*Smith, T. L., Co., 1125 32nd St., Milwaukee, Wis. (Harold E. Smith, Pres.)
Smoley, Constantine K. (International Text Book Co.), Scranton, Pa.

SMULSKI, EDWARD, 15 E. 40th St., New York City.
SNIDER, J. C., Camden, Ohio.
SNODGRASS, R. D., Youngstown, Ohio. (Truscon Steel Co.)
SOCIETE DES CIMENTS FRANCAIS, 80 Rue Taitbout, Paris, France. (F. Turquais.)
SORENSEN SONS & CO., JULIUS, 1805 West 6th St., Racine, Wis.
SOULE CO., EDWARD L., 916 Rialto Bldg., San Francisco, Cal. (Edward L. Soule.)
SOUTHERN CALIFORNIA EDISON CO., Edison Bldg., Los Angeles, Cal.
SOUTHERN CONCRETE PRODUCTS CO., Box 97, Anderson, S. C. (W. Carl Austin, Secy. and Treas.)
SOUTHERN STONE CORPN., 913 Healy Bldg., Atlanta, Ga.
SOUTHWESTERN PORTLAND CEMENT CO., 610-11 Mills Bldg., El Paso, Tex. (H. C. Swearingen.)
SPALDING, GEO. R., Lt. Col., Corps of Engineers, Wilson Dam, Florence, Ala.
SPAULDING, RALPH E., Apartado 1529, Lima, Peru. (Fred T. Levy & Co., Inc.)
SPELMAN, JOHN R., 405 Lexington Ave., New York City.
SPIKER & LOSE, 519-20 Forsyth Bldg., Atlanta, Ga. (William C. Spiker.)
SPINOSA, ARTHUR V., Braddock, Pa. (Consolidated Exp. Metal Co.)
SPOKANE CONCRETE PIPE CO., P. O. Box 1033, Spokane, Wash.
SPRAGUE, DERING J., 200 Fifth Ave., New York, N. Y. (Ballard Sprague & Co., Inc.)
SPRINGER, E. J., Rm. 1936, 30 Church St., New York City. (N. Y. Manager of Heine Chimney Co.)
SPRINGER, G. P., 514 E. Williams St., Apt. 1, Ann Arbor, Mich.
SPRINGER, G. P., 4816 Pine St., Philadelphia, Pa.
STACKHOUSE, A. V., 1118 Fletcher Trust Bldg., Indianapolis, Ind.
STAIRS, GORDON S., Town Manager, Wolfville, N. S., Canada.
STANAGE, JOHN L., 3526 Cedar St., Milwaukee, Wis.
STANDARD OIL OF NEW JERSEY, P. O. Box 37, Elizabeth, N. J. (Charles E. Haupt.)
STANDARD SLAG CO., Youngstown, Ohio. (F. E. Miller.)
STANTON, L. P., Villa Park, Ill.
STARR, GEORGE H., c/o Ferro Concrete Constr. Co., Cincinnati, O.
STEELE & SONS CO., WM., 219 N. Broad St., Philadelphia, Pa. (Edward A. Steele.)
*STEELE & SONS CO., WM., 219 N. Broad St., Philadelphia, Pa. (Edward A. Steele.)
STELLER, W. B., Box 264, St. Francis, Kan.
STEPHENS, A. W., 244 Madison Ave., New York City. (Turner Construction Co.)
STERN, I. F., 38 So. Dearborn St., Chicago, Ill.
STEVENS, FRANK B., JR., 800 Marquette Bldg., Detroit, Mich.
STEWARD & SILVER, 525 E. Hudson, Columbus, Ohio. (W. R. Steward.)

Stewart, Frank G., 1619-21 Sansom St., Philadelphia, Pa.

Stewart & Co., Inc., James, 30 Church St., New York City. (W. W. Lohmann, vice-pres.)

*Stone & Webster, 147 Milk St., Boston, Mass. (C. E. Nichols.)

Stonecrafters, The, 900 N. Main St., Ft. Worth, Tex. (E. A. LeMay.)

Storr, Albert E., 306 Chestnut St., Westfield, N. J.

*Straub, F. J., 1207 Victoria Ave., New Kensington, Pa.

Straub Patented Block Co., 409 N. Tod Ave., Warren, O. (Wm. I. Jennings.)

Struckel, Hermann, Mozambique Portland Cement Co., Ltd., Box 270, Lourenco Marques, East Africa.

Struckmann, Holger, c/o International Cement Corp., 342 Madison Ave., New York City.

Structural Service Bureau, 112 So. 16th St., Philadelphia, Pa. (D. Knickerbacker Boyd.)

Struthers, O. C., 720 Washington Bldg., Los Angeles, Cal.

Stubbs, Robert C., 1418 Pretorian Bldg., Dallas, Tex.

Stuhrman, Edward A., 762 Candler Annex, Atlanta, Ga.

*Superior Portland Cement Co., Seaboard Bldg., Seattle, Wash. (John C. Eden, Pres.)

Superior Portland Cement Co., Concrete, Wash. (C. L. Wagner.)

Superior Sand & Gravel Co., 925 Dime Bank Bldg., Detroit, Mich. (Russell E. Kean.)

Suter, Oscar M., c/o Vaccaro Bros. Co., La Ceiba, Spanish Honduras, Central America.

Sutherland, Hale, Mass. Institute of Technology, Cambridge, Mass.

Swain, George F., Harvard University, Cambridge, Mass.

Swan Company, The, 101 Tremont St., Boston, Mass. (E. Farnum Rockwood.)

Swansea Stone Works, R. R. 3, Belleville, Ill. (Arthur J. Fournie.)

Swanson, A. G., 28th Ave. and Sahler St., Omaha, Neb. (Omaha Concrete Stone Co.)

Swenson Construction Co., Shubert Theater Bldg., Kansas City, Mo. (Godfrey Swenson, Pres. and Treas.)

Syracuse Wall Plaster Co., Syracuse, N. Y. (John C. Johnson.)

Talbot, Prof. Arthur N., 1113 W. California Ave., Urbana, Ill.

Talbot, Kenneth H., Koehring Machine Co., Milwaukee, Wis.

Tamblyn, Gordon M., Room 9, Sweeney Bldg., 13th and Broadway, Denver, Colo.

Technische Hoogeschool, Onde Delft 69, Holland. (J. A. Bakker.)

Teknisk Bibliotek, Polyteknisk Laereanstalt, Kopenhagen K., Denmark. (Helge Holst.)

Tellyer, Harry B., 2810 N St., San Diego, Cal. (Cal. Glazed Cement Pipe Co.)

Thomas, A. O., 8540 Dexter Blvd., Detroit, Mich.

*Thompson & Binger, 103 Park Ave., New York City. (W. D. Binger.)

THOMPSON, JAMES, 811 Royster Bldg., Norfolk, Va.
THOMPSON, JOHN H., 15 Eastview Ave., White Plains, N. Y.
THOMPSON-LICHTNER CO., 136 Federal St., Boston, Mass. (Sanford E. Thompson.)
*THOMPSON-STARRETT CO., 51 Wall St., New York City. (N. M. Loney.)
THOMPSON, WILLIAM G., Bay Construction Co., St. Petersburg, Fla.
THURTLE, JOHN G., 3304 Flora Ave., Kansas City, Mo.
TIDEWATER PORTLAND CEMENT CO., Union Bridge, Md. (G. S. La Forge.)
*TIDEWATER PORTLAND CEMENT CO., Baltimore, Md. (B. T. Scott, Pres.)
TINKHAM CEMENT PRODUCTS WORKS, Palmetto, Fla. (M. E. Tinkham.)
TINKLER, J. W., 315 Snowden Ave., Bedford Pk., Toronto, Ont.
TOCH BROS., 110 E. 42nd St., New York City (Maximilian Toch.)
*TOCH BROS., INC., 110 E. 42nd St., New York City. (Maximilian Toch.)
TOMPKINS, CHARLES H., 1612 Park Road, N. W., Washington, D. C.
TORREY, JAMES E., 652 E. 26th St., Paterson, N. J. (John W. Ferguson Co.)
TOZZER, A. C., 178 Tremont St., Boston, Mass. (Turner Construction Co.)
TRAYLOR-DEWEY CONTRACTING CO., Allentown, Pa. (Willis Leriche.)
TREADWAY, H. P., 515 Orear-Leslie Bldg., Kansas City, Mo. (Kansas City Bridge Co.)
TREANOR, JOHN, 800 Corporation Bldg., Los Angeles, Calif. (Riverside Portland Cement Co.)
TRUFANT, RUSSELL ALDEN, Box 473, Metropolis, Ill. (Division of Highways.)
TRUMBULL, MARK M., c/o State Highway Commission, Raleigh, N. C.
*TRUSCON STEEL CO., Owen Bldg., Detroit, Mich. (S. M. Fechheimer.)
*TRUSCON STEEL CO., Youngstown, Ohio. (N. H. Sturdy.)
*TRUSCON STEEL CO., Youngstown, Ohio. (T. H. Kane.)
TRUSSED CONCRETE STEEL CO. OF CANADA, LTD., Walkerville, Ont., Canada. (F. P. Flett, Chief Engineer.)
TUBESING, WILLIAM F., Wauwatosa, Wis.
TURNER CO., C. A. P., 300 Bldrs. Exchange Bldg., Minneapolis, Minn.
TURNER, JOHN J., & SONS, Amsterdam, N. Y.
*TURNER CONSTRUCTION CO., 244 Madison Ave., New York City. (H. C. Turner.)
*TURNER CONSTRUCTION CO., 244 Madison Ave., New York City. (E. J. Moore.)
*TURNER CONSTRUCTION CO., 244 Madison Ave., New York City. (R. C. Wilson.)
*TURNER CONSTRUCTION CO., 244 Madison Ave., New York City. (T. A. Smith.)
UCHIMURA, SABRO, 45 South 5th St., Aoyama, Akasaka, Tokyo, Japan.
ULMAN, MALCOLM H., 317 So. Cameron St., Harrisburg, Pa. (State Dept. of Highways Lab.)
UNDERWRITERS' LABORATORIES, 109 Leonard St., New York, N. Y. (A. R. Small.)

United States Portland Cement Co., 305 Ideal Bldg., Denver, Colo. (Fred R. Schmidt.)

Universal Concrete Products Co., Box 118, New Martinsville, W. Va. (H. Eschenbrenner.)

*Universal Portland Cement Co., 210 S. LaSalle St., Chicago, Ill.

Universal Roofing Tile Co., 4113 Keeler Ave., N., Chicago, Ill. (E. L. Stephani.)

Upham, Charles M., c/o State Highway Comm., Raleigh, N. C.

Upson, Maxwell M., 140 Cedar St., New York City. (Raymond Concrete Pipe Co.)

Urquhart, L. C., Lincoln Hall, Ithaca, N. Y. (Cornell Univ.)

Usilton, L. H., c/o Barney-Ahlers Constr. Co., 110 W. 40th St., New York City.

Vandever, Jacob B., 1270 Broadway, New York City. (Nazareth Cement Co.)

Van Doorn, J. C., 836 Security Bank Bldg., Minneapolis, Minn. (Universal Portland Cement Co.)

Van Dyke, C. W., Asst. Gen. Mgr., Hampton Paint & Chemical Co., Springfield, Mass.

Van Scoyoc, H. S., Montreal, Que. (Canada Cement Co., Ltd.)

Verkouteren, A. J., General Delivery, Camden, N. J.

Viens, E., Director, Laboratory for Testing Materials, Public Works Dept., Ottawa, Ont.

Vigorose Cement Industrie, N. V., de Steeg, Holland. (B. A. Honig.)

Villadsen, James M., 303 Dooly Bldg., Salt Lake City, Utah. (Villadsen Bros. Inc.)

Villadsen, A. B., 303 Dooley Bldg., Salt Lake City, Utah. (Villadsen Bros., Inc.)

Vogel, John Leonard, 9 Hillside Ave., Box 336, Chatham, N. J. (New Jersey State Highway Commission.)

Voorhees, S. F., 342 Madison Ave., Room 2022, New York City.

Vortex Mfg. Co., 1978 W. 77th St., Cleveland, Ohio. (Chas H. Parkin.)

Voshell, James T. (U. S. Bureau of Public Roads), South Chicago Post Office Bldg., Chicago, Ill.

Voss, Walter C., Wentworth Institute, Boston, Mass.

*Vulcanite Portland Cement Co., Land Title Bldg., Philadelphia, Pa. (Jno. B. Lober, Pres.)

Vulcanite Portland Cement Co., 350 Madison Ave., New York City. (Albert Moyer.)

Waddell, William H., R. R. No. 2, St. Catherines, Ont.

Waddington, H. M., 156 Goodwill St., Rochester, N. Y.

Wadworth, John Frederick, 818 Commerce Bldg., Erie, Pa., P. O. Box 425.

Waldron, Edward M., Inc., 27 Central Ave., Newark, N. J. (W. J. Waldron.)

Walker, Frank R., 536 Lake Shore Drive, Chicago, Ill. (Frank R. Walker & Co., Inc.)

WALKER, STANTON, 1951 W. Madison St., Chicago, Ill. (Structural Materials Research Laboratories.)

WALLACE, W. K., Midland Ry., York Rd., Belfast, Ireland.

WALSH & BURNEY, P. O. Box 822, San Antonio, Texas. (R. L. Burney.)

WALKER, ERNEST, 263 Midland Ave., Montclair, N. J. (Porete Mfg. Co.)

WARD, A. A., c/o Ward-Kurtz Co., Inc., 415 E. State St., Ithaca, N. Y.

WARD, HARRY A., 11 Goodell St., Buffalo, N. Y. (Turner Const. Co.)

WARD, WM. B., 901 Fourth National Bank Bldg., Cincinnati, Ohio. (Tietig & Lee.)

WARNER CO., CHAS., 609 Morris Bldg., Philadelphia, Pa. (F. S. Lee.)

WARREN CO., F. V., 1913 Arch St., Philadelphia, Pa. (F. V. Warren.)

WARREN, GEORGE E., 111 W. Washington St., Chicago, Ill. (Portland Cement Ass'n.)

WARREN, SAMUEL, 25 Broadway, New York City. (Atlas Portland Cement Co.)

WASSEF, K. ZAKI, Bureau of Public Roads, Washington, D. C.

WASSER, T. J., 25 Kensington Ave., Jersey City, N. J.

WATSON CO., THE, 4614 Prospect Ave., Cleveland, Ohio. (W. J. Watson.)

WAUGH, H. L., Marquette Bldg., Detroit, Mich. (Albert Kahn.)

WAY, W. F., Northern Life Bldg., Seattle, Wash. (Henry & McFee.)

WEBB, JR., T. S., 1200 Island Home Pike, Knoxville, Tenn. (New Cast Stone & Tile Co.)

WEILLER, CHAS. A., 617 Lexington Bldg., Baltimore, Md. (Concrete Steel Co.)

WEINBERGER, A. J., Beach, N. D.

WEIR, C. LESLIE, 41 E. 42nd St., New York, N. Y. (American Ice Co.)

WEISS, PHILIP, 132 S. 7th St., Philadelphia, Pa.

WEITZ, FRED W., 713-15 Mulberry St., Des Moines, Ia. (Charles Weitz's Sons.)

WELLS & GRAY, LTD., 701 Confederation Life Bldg., Toronto, Ont. (Chas. Whittaker, Secy.)

WELLS, HEWITT, Woodward Bldg., Washington, D. C.

WELLS, P. A., 914 Monadnock Block, Chicago, Ill. (Wells Bros. Constr. Co.)

WELSCH, FRANK G., c/o Portland Cement Assn., Hubbel Bldg., Des Moines, Iowa.

WEMYSS, F. D., c/o A. S. Hall & Co., 71A Merchant St., Box 910, Rangoon, Burma, India.

WENDELN, C. F., 542 Milton St., Cincinnati, Ohio.

WENZEL, JR., HERMAN, 201 S. Culver St., Chippewa Falls, Wis. (Chippewa Cement Block Works.)

WEST, C. W., Welland Ship Canal, Thorold, Ont., Canada.

WESTERGAARD, HAROLD M., University of Illinois, Urbana, Ill.

WESTINGHOUSE ELECTRIC MFG. CO., E. Pittsburgh, Pa. (J. E. Webster.)

WESTON, CLARENCE M., Room 1303, 200 Fifth Ave., New York, N. Y. (H. S. Ferguson.)

WETSTEIN, MENTOR, 1205 Harrison Ave., Cincinnati, O. (Concrete Surfacing Mchry. Co.)

WHEELER, EARL J., c/o Wells Bros. Construction Co., 2825 E. 14th St., Oakland, Calif.

WHEELER, J. A., c/o Texas Portland Cement Co., Dallas, Texas.

WHEELER, WALTER H., 1112 Metropolitan Life Bldg., Minneapolis, Minn.

WHITE & CO., BARCLAY, 1713 Sansom St., Philadelphia, Pa. (Barclay White.)

*WHITE CONSTRUCTION CO., INC., 95 Madison Ave., New York City. (G. Edward Escher.)

WHITE SUN DIAL CO., 1332 Union Ave., Memphis, Tenn.

*WHITEHALL CEMENT MFG. CO., 1736 Land Title Bldg., Philadelphia, Pa. (P. D. Gaskill.)

WHITNEY BROS. CO., 1st and Baxter, Superior, Wis. (Gwin A. Whitney.)

WHITNEY, CHAS. S., 306 Colby & Abbot Bldg., Milwaukee, Wis. (Hoe & Johnson.)

WICKERSHAM, JOHN H., 53 Duke St., Lancaster, Pa.

WIDMER, ARTHUR J., 941 Century Bldg., St. Louis, Mo. (Widmer Engineering Co.)

WIGHTMAN, S. H., Monadnock Block, 53 W. Jackson Blvd., Chicago, Ill. (Besser Sales Co.)

WIGTON, C. B., 552 W. 22nd St., New York City. (Wigton-Abbott Corp.)

WILDER, A. D., 605 Metropolitan Bldg., Vancouver, B. C., Canada. (Sydney E. Jenkins Co.)

WILES, J. R., 8 W. 40th St., New York City.

WILHELM, GEO. T., Cedar Rapids, Iowa. (Geo T. Wilhelm Co.)

WILL, CHARLES H., P. O. Box 404, Hilo, Hawaii.

WILLIAMS, CHARLES H., Box 126, Toledo, Wash. (State Highway Dept.)

WILLIAMS, G. M., University of Saskatchewan, Saskatoon, Sask, Can.

WILLSON, S. H., 514 Merchants National Bank Bldg., St. Paul, Minn. (Kalman Floor Co.)

WILSON, FRANCIS F., 3 Wilson Lane, Rockville Centre, N. Y.

WILSON, JOS. P., 8026 Moffatt St., Detroit, Mich.

WILSON & CO., R. F., 1851 Elston Ave., Chicago, Ill. (R. F. Wilson.)

WILSON, W. A., Roswell, N. Mex.

WILSON'S (N. Z.) PORTLAND CEMENT, LTD., Box 269, G. P. O., Auckland, N. Z.

WINELL, V. E., 51 Wall St., New York, N. Y. (Thompson-Starrett Co.)

WINSTON BROS. CO., 801 Globe Bldg., Minneapolis, Minn. (W. H. Gardiner.)

WISCONSIN UNIVERSITY LIBRARY, Madison, Wis. (W. M. Smith.)

WISEMAN, W. J., 1942 Forbes St., Pittsburgh, Pa. (Booth & Flinn, Ltd.)

WITHEY, MORTON O., 1921 W. Lawn Ave., Madison, Wis. (University of Wisconsin.)

WITT, J. C., 881 Cornelia Ave., Chicago, Ill.

WITT, W. H., 204 Seaboard Bldg., Seattle, Wash.

WOLPERT, OTTO, 51 Wall St., New York City. (Thompson-Starrett Co.)
WOLVERINE PORTLAND CEMENT CO., Coldwater, Mich.
WOODLE, JR., A. S., c/o The Baldwin Locomotive Works, Philadelphia, Pa.
WOODLEY & GRANGER CONSTR. CO., Benton Harbor, Mich. (Louis J. Woodley.)
WOODS, HARLAND C., 442 Grove St., E. Lansing, Mich.
WOOLF, DONALD O., Bureau of Public Roads, Washington, D. C.
WOOLSON, IRA H., 76 William St., New York City. (National Board of Fire Underwriters.)
WORCESTER, J. R., 79 Milk St., Boston, Mass.
WORCESTER, R. J. H., 804 E. North St., Greenville, N. C.
WRIGHT, EVERETT G., c/o Albert Kahn, Marquette Bldg., Detroit, Mich.
WRIGHT, NELDIEN H., Box 313, West Jackson Sta., Jackson, Miss.
WRIGHT & KOWALSKI, 15 Exchange Pl., Jersey City, N. J. (Donald A. Wright.)
WU, POND S., Supervisor of Works, Govt. Salt Administration, Peking, China.
*WYANDOTTE PORTLAND CEMENT CO., Ford Building, Detroit, Mich. (John W. Boardman, vice-pres.)
YAKIMA PAVING CO., 324 Perkins Bldg., Tacoma, Wash. (Geo. Milton Savage, Pres.)
YANDELL & CONGER, E. Lafayette St., Jackson, Tenn. (C. H. Yandell.)
YINGLING, JR., CLINTON K., 1105, 17th St., N. W., Washington, D. C. (Municipal Architects Office.)
YORK PATENT BLDG. BLOX CO., Cor. Chestnut St. and P. R. R., York, Pa. (C. C. Spahr, pres.)
YOUNG, CHARLES H., 822 McKnight Bldg., Minneapolis, Minn.
YOUNG, CLARENCE R., 98 Hilton Ave., Toronto, Ont. (University of Toronto.)
YOUNG, R. B., 8 Stracham Ave., Toronto, Ont. (Hydro Electric Power Comm.)
YOUNG, R. N., Bureau of Standards, Washington, D. C.
YOUNG, R. W., Paullina, Iowa. (Paullina Con. Silo Co.)
YOUNG, THOMAS F., 8 Great George St., Westminster, London, S. W. 1, England.
YOUNGSTOWN BURIAL VAULT CO., 546 E. Indianola Ave., Youngstown, Ohio. (E. E. King.)
ZEIDLER, JOHN L., 401 Francis St., St. Joseph, Mo. (Zeidler Concrete Pipe Co.)
ZIEGLER, A. WARREN, Ferro Concrete Construction Co., Cincinnati, Ohio.
ZIPPRODT, ROY R., 410 Grain Exchange Bldg., Sioux City, Iowa.

INDEX